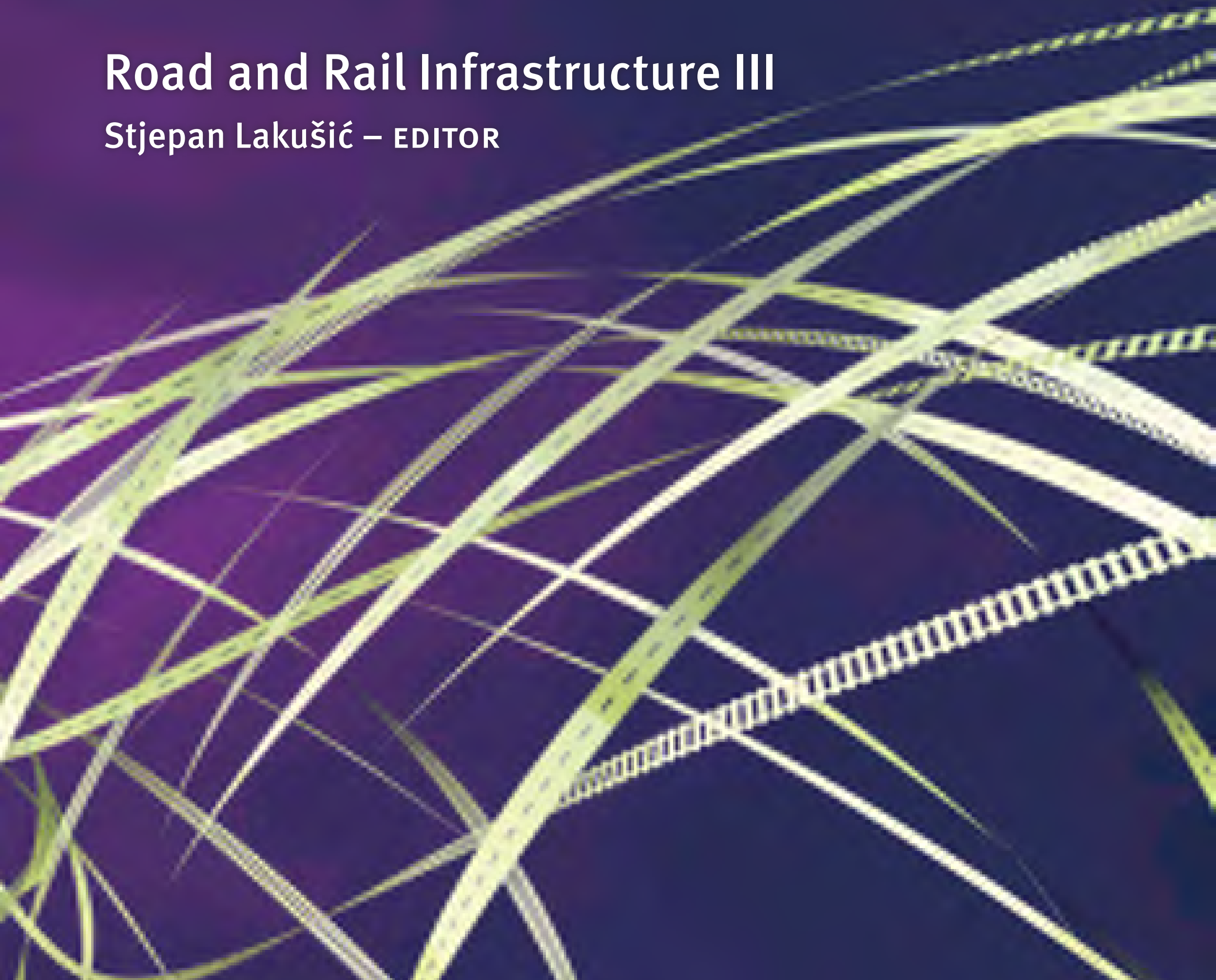
Road and Rail Infrastructure III

CETRA 2014

3rd International Conference on Road and Rail Infrastructure
28–30 April 2014, Split, Croatia

Road and Rail Infrastructure III

Stjepan Lakušić – EDITOR



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Road and Rail Infrastructure III

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FOREWORD

The 3rd International Conference on Road and Rail Infrastructure – CETRA 2014 was organized by the University of Zagreb - Faculty of Civil Engineering, Department for Transportation Engineering. The Conference was held in Split, Croatia. Split is the largest city in Dalmatia and the second largest city in Croatia, and it is also one of "Croatian Champions of Tourism".
The 1st International Conference on Road and Rail Infrastructure (CETRA 2010) was held on 17-18 May 2010 in Opatija. The 2nd International Conference on Road and Rail Infrastructure (CETRA 2012) was held on 7-9 May 2012 in Dubrovnik. A great interest of participants in topics and themes from the field of road and rail infrastructure, as shown during the CETRA 2010 conference (140 papers from 29 countries) and CETRA 2012 conference (142 papers from 39 countries), justified the Department of Transportation Engineering's decision to organise once again an international event of such great significance. Positive comments received from participants in past conferences motivated the Department for Transportation Engineering of the Faculty of Civil Engineering - University of Zagreb to continue with the organization of this international event.
The CETRA conference has established itself as a venue where scientific and professional information from the field of road and rail infrastructure is exchanged. The idea on linking research organisations and economic operators has been the guiding concept for the realisation of this conference. Conferences of this kind are undoubtedly a proper place for bringing closer together the economy and university operators, and for facilitating communication and establishing greater confidence that might result in cooperation on new projects, especially those that contribute to greater competition. Lectures organized in the scope of the conference are based on interesting technical solutions and on new knowledge from the field of transport infrastructure as gained on already realised projects, projects currently at the planning stage, and those now under construction, in all parts of the world. In addition to authors from the academic community, lectures were also presented by practical authors, the idea being to ensure the best possible synergy between the theory and practice. Because of a great interest for the themes from the field of road and rail infrastructure, as shown during the past two conferences (CETRA 2010 and CETRA 2012), the Department for Transportation Engineering of the Faculty of Civil Engineering – Zagreb assumed the responsibility to organise the CETRA conference in this year as well.
Our goal for the International Conference on Road and Rail Infrastructure – CETRA is to have all published papers indexed in scientific databases in order to achieve greater recognition for the conference itself, for published papers, and for their authors. As the serial publication entitled Road and Rail Infrastructure has been achieved with this third conference, the precondition has been fulfilled to obtain the International Standard Serial Number (ISSN), which was the condition for starting procedure for registering this publication in scientific databases. The procedure has already been initiated.
The third International Conference on Road and Rail Infrastructure – CETRA 2014 - is organised in this year in order to bring together scientists and experts from the fields of road and railway engineering, and to present them with yet another opportunity to share results of their research, findings and innovations, analyze problems encountered in everyday engineering practice, and offer possible solutions for a more efficient planning, design, construction, and maintenance of various transport infrastructure facilities and projects.
CETRA 2014 covers many areas: traffic planning and modelling, infrastructure projects, infrastructure management, road pavements, rail track superstructure, construction and

maintenance, transport geotechnics, tunnels and bridges, structural monitoring and maintenance, computer techniques and simulations, noise and vibration, innovation and new technology, urban transport, integrated timetables on railways, rail traffic management systems, vehicle dynamics, traffic safety, and bicycle traffic.
CETRA 2014 attracted a large number of papers and presentations from 35 countries and 47 universities. More than 146 papers were presented at the conference and are grouped together in these proceedings entitled Road and Rail Infrastructure III. The papers are conveniently divided into twelve chapters: Rail Infrastructure Projects Design, Construction, Maintenance and Management, Road Infrastructure Projects Construction, Maintenance and Management, Road Traffic Planning and Modelling, Road Pavements, Rail Vehicle-Track Interaction, Structural Monitoring and Maintenance, Transport Geotechnics, Integrated Timetables on Railways, Traffic Safety, Environmental Protection, Urban Transport and Passenger services: baggage storage and boarding.
The organizers of the conference wish to express their thanks to all businesses and institutions that provided their valuable support to this Conference. Special thanks are extended to the University of Zagreb, Croatian Railways – HŽ Infrastruktura, and Ministry of Maritime Affairs, Transport and Infrastructure, for their assistance in organizing the workshop on Implementation of European Rail Traffic Management System (ERTMS) in South and East Europe.
The Editor commends all authors for excellent papers contributed to these proceedings, and wishes to thank members of the International Academic Scientific Committee, and numerous experts who participated in the review process. The gratitude is also extended to all participants for deciding to come to Split and take part in CETRA 2014. We believe that these CETRA 2014 proceedings entitled Road and Rail Infrastructure III will be, just like the preceding two proceedings from the CETRA cycle, highly interesting and useful to all experts exhibiting a scientific and professional interest in road and rail infrastructure.

THE EDITOR
Prof. Stjepan Lakušić
April, 2014.

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KEYNOTE LECTURES

GEOTECHNICAL CHALLENGES FOR THE EUROPEAN TEN-T NETWORK – SMARTRAIL AND BEYOND

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Abstract

Climate change is leading to increased incidence of slope instability across the globe. As well as causing a risk to human life, slope failures can cause severe disruption to transport networks. Infrastructure managers who control road and rail networks need to manage risks associated with earth slopes, many of which were built before the advent of modern design and construction standards. This paper discusses two key steps in the slope management process: (i) locating potential failures and (ii) rational analysis of slopes. Recent work completed in the EU FP7 SMARTRAIL project is described and a case study is presented demonstrating how the potential damaging effects of rainfall can be considered in a rational framework.

Keywords: infrastructure, climate change, risk assessment, slope modelling

1 Introduction

EU transport policy provides the challenge to infrastructure managers (IMs) to increase the productivity of existing transport networks, prioritise renewal and optimise new sections to reduce bottlenecks, increase productivity and achieve a switch from freight transport by road to rail. This needs to be achieved at a time when budgets are restricted, whilst improving customer satisfaction and dealing with challenges from both natural hazards and extreme weather events, which are affecting all of Europe, see Figure 1.
Whilst great strides in providing new motorways, designed and built to modern standards, has been achieved using EU structural funds. Rail (IMs) are managing ageing rail infrastructure with 95% of the network having been built before 1914. IMs across Europe are spending a total in the region of €15 to 25 billion annually on railway infrastructure maintenance and renewal (European Commission 2013). Much of the existing road and rail infrastructure throughout the EU was built before the advent of modern design and construction standards, management of these assets presents a particular challenge.
The current response to infrastructure failure is reactive i.e. when failures occur they are fixed. The location of the failure then becomes a hot-spot on the network. Forensic analyses of these failures often reveal that indicators of distress were ignored due to a lack of understanding or an absence of a proper framework for decision making. While asset management systems and risk assessment frameworks are ubiquitous across Europe and the world, there is an over reliance on visual assessment and subjective forms of risk rating. In addition important information on asset performance over a long period tends to be stored either locally or even with individuals. Loss of corporate memory has been identified as one of the key reasons for the catastrophic failure of the Malahide Viaduct on the Irish TEN-T Network in 2009. Significant research effort is being applied to improving the current management of transport infrastructure.

The FP7 project SMARTRAIL-Safe Maintenance Analysis and Remediation of rail Transport infrastructure (www.smartrail/ferhl.org) is an EU funded project. The objective of the research project is to use the latest monitoring technology and IT networks to provide instantaneous data on structural performance which can be used to undertake real-time assessments of railway infrastructure such as embankments, bridges and tunnels. In parallel, effective and cost efficient remediation techniques are being investigated. These engineering assessments of current state will be used to design remediation strategies to prolong the life of existing infrastructure in a cost-effective manner with minimal environmental impact. Whilst the project is wide ranging, in this paper we concentrate on the current methods of assessing risk along transport networks. In particular, methods developed in SMARTRAIL for improving monitoring and assessment of rainfall induced slope failures is considered in this paper.

Figure 1 Examples of infrastructure challenges in Europe

2 Risk assessment

While a number of risk assessment and management methods have been proposed, they all follow a similar structure on a macro scale. Central to the meaningful performance of risk assessment at a local or network level is the performance of hazard assessments. In this instance these deal with characterization of landslide events and the determination of the probability of occurrence. Risk management, as an interdisciplinary infrastructure management tool, is now a well-established field with National and European Guidance and procedures (e.g. PAS 55). Recent advances in GIS and other software have made it possible to implement risk management tools over large geographical areas for linear infrastructure networks, See Figure 2. Thus making them a highly usable tool for infrastructure managers and stakeholders.

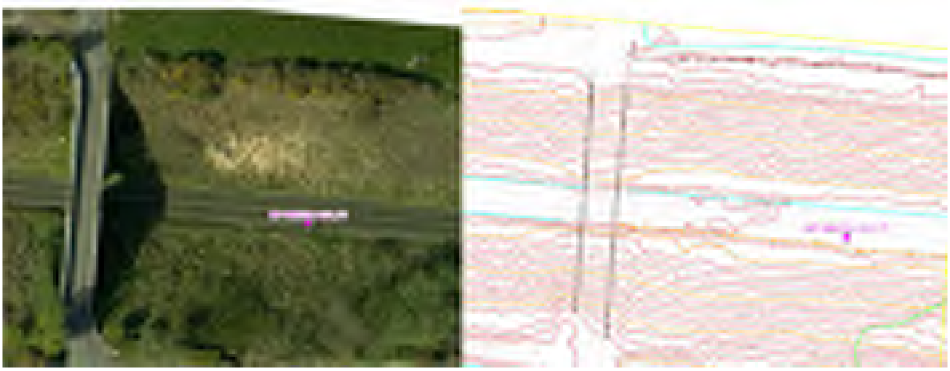

Figure 2 Example of various dataset layers obtained through LiDAR survey

Whilst a variety of approaches can be adopted to determine the hazard assessment, e.g. fault tree analysis, risk matrices etc. this central determination is predominantly based on data collected from visual assessments. Whilst observation of the condition of an element of infrastructure by a suitably qualified and experienced person is an important aspect of the overall life-cycle management, the result of any inspection is by its nature subjective and qualitative. The aim of the SMARTRAIL project is to move towards less subjective considerations using the FACT principle (Find, Analyse, Classify and Treat). This paper describes recent research aimed at improving how we find and monitor potential failures and having established the location of problem areas, the procedures which should be followed in quantifying the level of risk.

2.1 Locating hot-spots

An important part of the process of management of infrastructure networks is prioritisation of areas for maintenance and upgrading. The first step in this process is to identify at-risk objects. Whilst visual assessment is a useful tool in the assessment of the condition of slopes, it should not be the principal method for investigation as processes controlling slope stability are often not evident (e.g. changes in moisture content or the presence weak zones/lenses), and visual evidence of distress often occurs when it is too late to stop the process. An example of the limitations of performing visual slope assessments on a 300 m embankment, Inch embankment, at a site in South-East Ireland is described. The embankment in question is between 5 m and 6 m high and has a relatively steep side slope of ≈ 45°. The embankment had suffered a failure during installation of a cable at the top of the slope about 10 years before the inspection described here. The section where the failure occurred was remediated by regrading the slope using a geotextile reinforced berm, See Figure 3 (the berm is visible to the left of the photo taken from a train passing over a bridge at the start of the embankment). Because of the past failure in the area, the section of the remaining embankment was classified as a hot-spot, i.e. an area at high risk of failure. Because of the remedial work which had been performed at the failure site, this area was not considered to be at risk and was excluded from the walk-over visual survey. The visual inspection reported that the visual slope condition, vegetation, adjoining land sections etc. were indistinguishable along the entire 300 m section of the embankment and therefore the potential risk of failure for the remaining embankment was high. A geophysical survey of the site was recommended. Within six months of the visual assessment having been performed a slope failure occurred during a period of heavy rainfall and the line was closed for a number of days to allow an investigation to be undertaken. The failure occurred in the same area as the previous failure. To investigate the cause of the failure a geophysical investigation which included Multichannel Analysis of Surface Waves (MASW) and a geotechnical investigation was performed. Following interpretation the MASW technique allows a 1D profile of shear wave stiffness (V_s) to be derived and this can be used to infer the small strain shear modulus of the soil (G_0), Donoghue et al. (2003). The results of this investigation (See Figure 3) revealed that the embankment fill used to construct the first 100 m section of the embankment had much lower stiffness magnitudes than the remaining 200 m section. This would suggest strongly that the material in the first section is significantly weaker and given that the geometry of the embankment is similar along the entire length the first section is at much higher risk of failure.

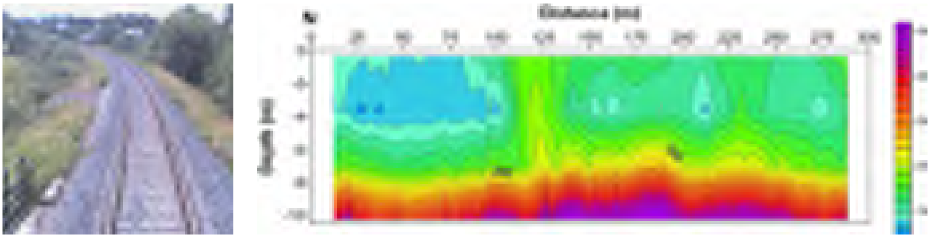

Figure 3 a) Photo of Inch Embankment, b) Shear Wave Velocity Measurements in m/s (scale on right)

The second failure considered was located in Enniscorthy, in the South-East of Ireland. The railway embankment, which was built in 1850's had a history of poor serviceability, with regular re-ballasting required. Because of these on-going settlement issues coupled with a requirement to increase the shoulder width at the top of the embankment, a plan to re-model the embankment was implemented in late 2006. During the reconstruction of the embankment a period of heavy rainfall was experienced and a large slope failure occurred. The failure mode was rotational with the failure surface cutting into the natural ground on the river-side of the embankment toe. An investigation was commissioned to determine the cause of the failure. The first phase of this investigation consisted of drilling four, shell and auger boreholes perpendicular to the failure surface (see Figure 4). The boreholes revealed that the ballast thickness at the failure surface was greater than 1 m. The embankment fill was comprised of soft to firm silty clay (e.g. with undrained strengths in the range 40 to 75 kPa based on correlations to SPT N). The soil at greater than 2 m below original ground level was a glacial till which covers much of Ireland. This glacial soil was typically 2 m thick and was described as stiff (undrained strength > 150 kPa) and overlaying Siltstone bedrock. BH 4 which was located at the river side of the embankment contained a 1 m thick layer of peat between the upper silty clay and the lower glacial soil. The large depth of ballast evident at the failure location was in keeping with observations of the local railway staff that the section of track required constant re-ballasting and although the embankment height was relatively constant, the failure location was at a low point on a vertical curve. There was some suspicion therefore, that the thick ballast layer may have acted like a drain and funnelled water to the location at which the failure occurred.

The authors initiated a geophysical investigation and additional sampling and laboratory tests followed this. The range of geophysical techniques used, included Ground Penetrating Radar (GPR), Electrical Resistivity Tomography (ERT) and Multichannel Analysis of Surface Waves (MASW). The GPR was used specifically to check the ballast thickness approaching the failure section from both directions along the track, the ERT was used to profile the underlying strata, whilst the MASW was used to obtain stiffness data for the strata. The most recent surficial ballast layer was shown to be approximately 0.5 to 0.6 m thick, and was generally consistent throughout the length of the profile. Below this, an interesting feature in the GPR profile occurred between 110 and 180m where the ballast layer appears to thicken considerably over a very short distance to a depth of over 2 m. The presence of thicker ballast in this area suggests a long-term settlement problem. The reason for this problem became apparent when the ERT profile was interpreted.

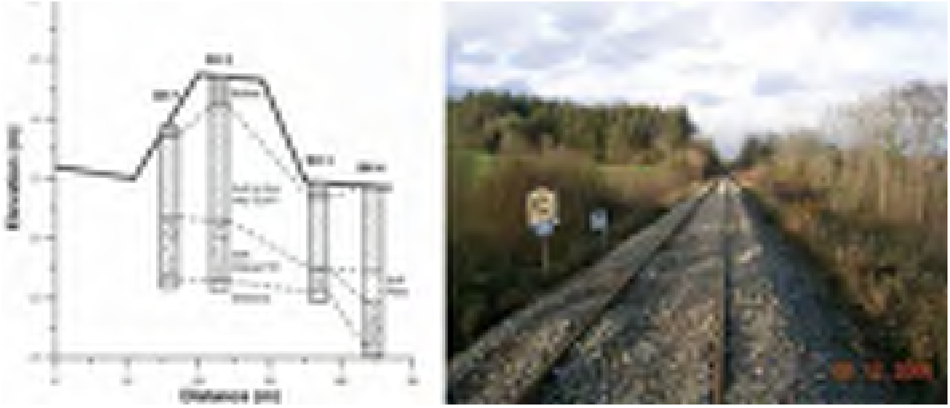

Figure 4 a) Boreholes taken across failure, b) Photo of Enniscorthy Embankment

A section ERT2 (See Figure 5) running along the track with the failure surface approximately at the centre, shows the presence of extremely soft soils (low resistivity shown by red colour) below the embankment, which most likely was part of an ancient river channel. The soft soil layer was up to 8 m thick at the location of the failure. Sample of this soil were recovered for laboratory testing. They revealed that the undrained strength, s_u of the material was ≈ 22 kPa. Construction of the remodelled embankment resulted in the application of an applied stress of 88 kPa. Simple bearing capacity calculations (Donohue et al. 2010) suggested that the shear strength of the underlying soil was marginal. However, odometer tests found that significant and long-lasting creep strains could be expected under the applied load.

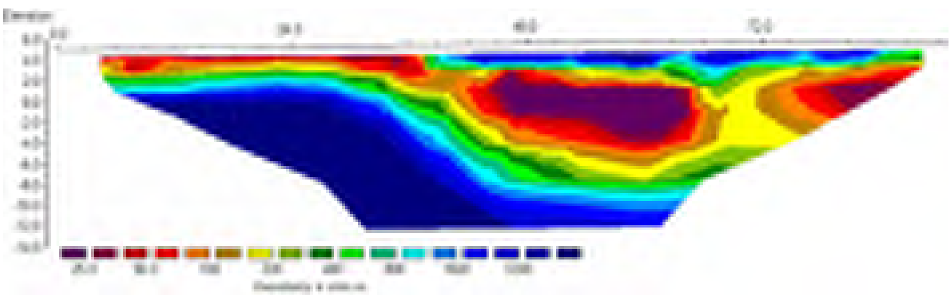

Figure 5 Resistivity measurements made along the track. The rail level is at 6 m, the base of the embankment is at 2 m. The presence of deep soft deposits under the embankment at the failure location (48 m along track) is evident

The above examples show that simple and low-cost geophysical techniques can provide very clear indications of problems locations along linear transport networks. Many rail IM's have collected GPR data along their entire network as a quality assurance measure for ballast maintenance. It is suggested that unusual variations in ballast thickness might be used as means of detecting problem locations along the network that should be investigated further. If a problem is identified one of the most useful means of managing the risk is to provide structural health monitoring. The SMARTRAIL project (2013) reviewed and developed methods for monitoring critical infrastructure considering; bridge scour, slope stability, bridge corrosion, sun-soil and ballast assessments. Whilst these are not reviewed further in this paper, the reader is directed to www.smartrail.ferhl.org for details. One of the benefits of collecting SHM data along a network is that is can feed statistical data to a probabilistic based assessment of infrastructure performance. A topic considered in the following section.

3 Slope stability analysis

A large proportion of landslides that occur along transport networks occur during or shortly after periods of heavy rain. In man-made embankments and cuttings where the water table level is controlled, negative pore water pressure (suctions) develop in the near surface soils. These suctions contribute significantly to the slopes overall stability. However, these suctions are transient and during periods of rainfall, infiltration of water causes an increase in moistu-re content and a reduction in suction. The near surface zone affected by water infiltration is known as the wetting front (zone of decreased suction), see Figure 6. Fredlund and Rahardjo (1993) and Fourie et al. (1999) note that decreased suction in this zone results in a reduction of the total (or apparent) cohesion of the soil, and therefore it's strength, whilst at the same increasing the self-weight of the soil (and thus the destabilising force).

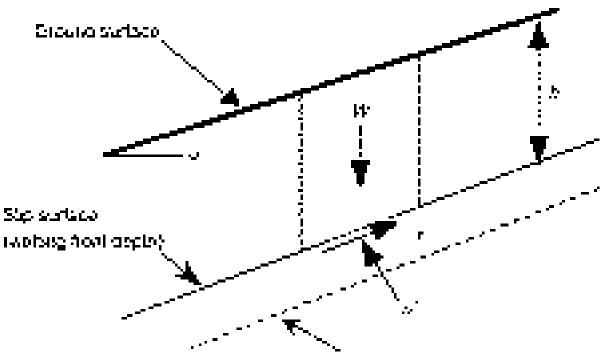

Figure 6 Planar failure surface for rainfall induced landslides

3.1 Effect of suction

the Analysis of the stability of unsaturated slopes is a complex problem, however, simplified limit equilibrium analyses have been developed. Fredlund et al. (1978) expanded the Mohr-Coulomb model to incorporate negative pore pressure, suction effects:

$$\tau = c' + (\sigma_n - u_a)\tan\phi' + (u_a - u_w)\tan\phi^b \tag{1}$$

Where:

τ	shear strength;
c'	effective (apparent) cohesion';
σ_n	total normal strength on the failure plane;
u_a	pore-air pressure on the failure plane;
ϕ'	angle of internal friction associated with the net normal stress state variable σ_n-u_a;
u_w	pore-water pressure on the failure plane;
u_a-u_w	matric suction on the failure plane;
ϕ^b	angle controlling the rate of increase in shear strength relative to the matric suction.

The expression can be simplified to the conventional form of the Mohr-Coulomb Equation by combining the effects of c' and the contribution of matric suction (u_a-u_w) tanΦ^b into a single parameter (C).

$$\tau = C + (\sigma_n - u_a)\tan\phi' \tag{2}$$

Since in rainfall induced slope failures the failure plane forms parallel to the slope, Fourie et al. (1999) suggest using an infinite slope model in which the soil strength is described by an expression of the form given in Eq 3, to analyse this failure mechanism. The Factor of Safety (F) is given by:

$$F = \frac{c' + (u_a - u_w)\tan\varphi^b + \gamma h \cos^2\alpha \tan\varphi}{\gamma h \cos\alpha \sin\alpha} \tag{3}$$

Where:

γ unit weight of soil;
h wetting front depth;
α slope angle.

Whilst the expression allows the factor of safety to be calculated in a deterministic analysis many of the input parameters needed for these analyses are highly variable. The result of any such analysis therefore is critically dependent on assumptions made by the designer. Traditionally the designer assumes some constant (low) value for total cohesion and then by varying the wetting front depth within a reasonable range, an estimate of the factor of safety is determined.

3.2 Reliability analysis

in reality the parameters, such as wetting front depth, suction and rainfall intensity are highly variable and lend themselves to probabilistic analyses. A number of workers, Hassan and Wolff (1999), Chowdhury and Grivas (1982), Christian et al. (1994), Low (2003); Low and Tang (1997), and others have proposed reliability based approaches to analyse slope stability problems. However, primarily due to mathematical complexities which arise due to the non-linear form of performance functions typically used in geotechnical design, many of these are hybrid approaches which for example use deterministic methods to locate the critical slip surface of a slope with fixed shear strength parameters and then expand the variables probabilistically about this surface. Xue and Gavin (2007) and Gavin and Xue (2009) describe a model which is capable of simultaneously locating the critical slip surface and determining the reliability of a slope. By transforming the variables such as cohesion, wetting front depth and slope geometry into polar coordinates the complexities associated with defining the limit state function were overcome. In addition this minimisation problem was solved using a powerful and efficient Genetic Algorithm approach which was shown to avoid the numerical errors associated with the cosine directional approaches used by many workers which tend to identify non-minimum reliability indices for non-linear functions. For mathematical convenience the method developed by Xue and Gavin (2007) known as Genetic Algorithm for Slope Stability Analysis (GASSA) assumed normal distributions for variables. For geotechnical parameters where normal distributions are not always reasonable useful limits on parameter values had to be imposed. Reale (2015) addressed this deficit by developing an updated code which allowed a range of more realistic distributions to be considered in GASSA in work funded through the SMART RAIL project.

4 Case study

4.1 Background

To investigate the effect of analysis type (deterministic versus probabilistic) on the predicted safety of an embankment slope, a study of the effect of rainfall on the stability of a Victorian era railway embankment was performed. The 7.5 m high embankment is located in Co. Meath, Ireland (See Figure 7). The embankment has a relatively steep slope angle of approximately 38° which is typical of Irish Railway embankments (Jennings and Mulddon 2003). It is constructed from a glacial till, with low fines content (less than 20%). The natural moisture content of samples taken from within the embankment and measurements of in-situ water content measured over a five month period revealed moisture contents that ranged from 17.4% to 23.5%.

Figure 7 a) Location of embankment shown by red dot, b) photo of embankment preparation for instrumentation

A simple laboratory experiment was performed on the embankment fill to investigate the effect of increasing moisture content on suctions(Reale et al 2012). Samples of the soil were compacted into standard proctor moulds and were then ponded (i.e. a constant head of water was applied to the top surface). A tensiometer, placed at the base of the mould, allowed the variation in suction with time to be determined. The results of tests on two samples, which were compacted at initial moisture contents of 15% and 20% respectively, are shown in Figure 8. The initial suction varied from 20 kPa for the sample with the highest moisture content to 40 kPa for the drier sample. As water infiltrated into the samples the suction decreased rapidly. The sample with the lowest initial water content, and higher initial suction allowed water to infiltrate at a higher rate, and thus suctions reduced faster in this sample initially. However, after approximately 12 hours of infiltration the suction in both samples was equal. The suctions continued to decrease over the period of the test (40 to 50 hours) and approached a residual value of between 3 and 4 kPa. Tensiometers placed in the slope revealed that suctions monitored over a five-month period varied between 4 kPa and 9 kPa (September 2013 to January 2014).

4.2 Deterministic Assessment

The factor of safety, F of the slope can be determined using Eq 3. Since the model assumes a planar failure surface develops which is controlled by the wetting front depth, h this parameter was varied in the analyses to quantify the effect of increasing wetting front depth during a rainfall event. All other parameters were assigned the constant values shown in Table 1. The soils unit weight and constant volume friction angle were measured in the laboratory. The average suction measured during the five month monitoring period was used as the suction value for design, whilst the Φb value was chosen based on guidance in Gan et al. (1988).

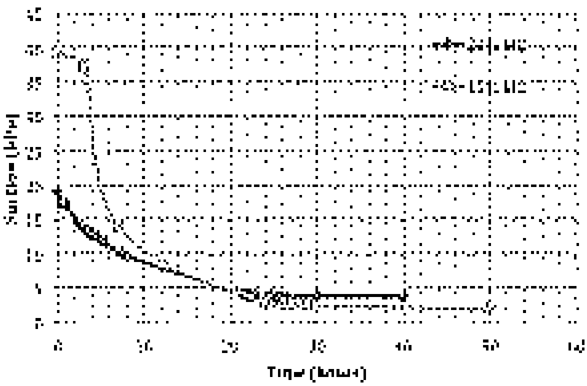

Figure 8 Laboratory infiltration for the embankment fill

Table 1 Deterministic parameter values

Parameter	Value
Cohesion (c')	0
Soil suction (u_a-u_w)	7 kPa
Internal angle of friction	34°
Rate of increase in shear strength due to matric suction Φ^b	24°
Slope angle	38°
Unit weight	17 kN/m^3
Wetting front depth	Varied from 0.1 m to 2 m

By varying the wetting front depth from 0.1 m to 2 m the factor of safety reduced towards the critical value of one, but did not reach it suggesting some reserve of strength. See Figure 9. As the vast majority of shallow embankment failures in Ireland occur in the top 1.5 m of the soil, this is a positive result.

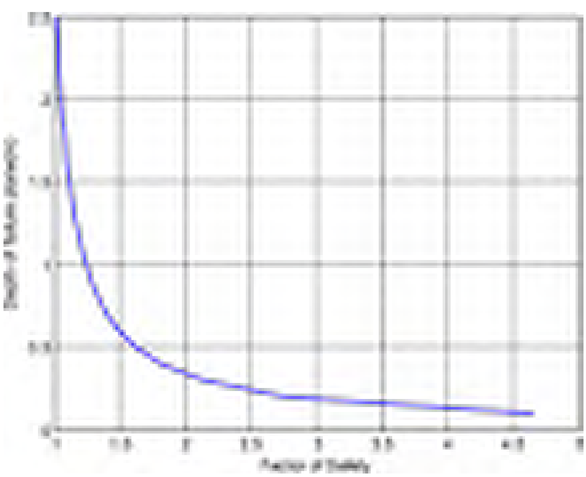

Figure 9 Effect of wetting front depth on factor of safety from deterministic analysis

It should be noted however that the use of average values might distort the analysis. Considering the histogram of suction values measured during the monitoring period (see Figure 10) it can be seen that whilst the mean suction value was 7 kPa, there was substantial variation with a coefficient of variation of approximately 0.2. As a result the factor of safety derived from the mean suction value may be a poor indicator of actual stability. If the suction value used for the analysis is reduced to 3 kPa (i.e. the residual value measured in the laboratory experiment), failure is predicted at a wetting front depth of 1m. The time required to achieve such a low suction over the entire wetting front depth of 1 m would be substantially longer than that observed in the lab due to the much longer drainage path length. Given the rainfall patterns experienced in Ireland and evidenced from the suction measurements on site, such low values are unlikely in the near future.

4.3 Probabilistic analyses

The GASSA model was used to determine the reliability index, β with the performance function, g(x):

$$g(x) = \frac{c' + (u_a - u_w)\tan\varphi^b + \gamma h \cos^2\alpha \tan\varphi'}{\gamma h \sin\alpha \cos\alpha} - 1 \tag{4}$$

The model requires all random variables to be transformed into a standard normal space before calculating the reliability index β. If the parameters are uncorrelated this can be easily accomplished by the following equation.

$$\overline{x}_i = \frac{x_i - \mu_i}{\sigma_i} \quad \text{for } i = 1,2,\ldots,n \tag{5}$$

Where:

x_i	vector representing the entire set of random variables;
$\overline{x}_i$	reduced set of random variables in the normal space;
μ_i	vector of the parameter means;
σ_i	vector of the parameter standard deviations.

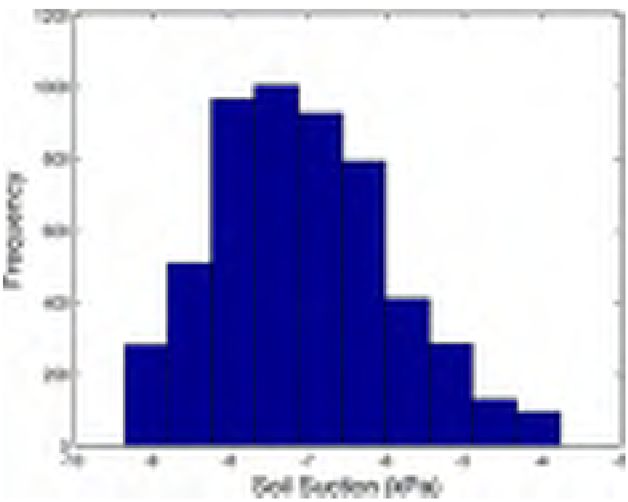

Figure 10 Histogram of suction measurements taken from embankment over five month monitoring period

If the parameters are non-normal they must be transformed into equivalent normal distributions first, using the Rackwitz-Fiessler (1978) two parameter equivalent normal transformation. Once equivalent normal distributions are obtained the parameters can then be transformed into the standard normal space using the equation outlined above. A normal distribution was assumed for the angle of internal friction and the rate of increase of shear strength due to matric suction. Lognormal distributions were assumed for soil suction and soil unit weight. The glacial till soils used to form the majority of earthworks in Ireland have no natural cohesion so a deterministic value of zero for c' was assumed. After transforming the variables to the standardised normal space the limit state function can be rewritten as:

$$g(X) = g(\overline{x_1}, \overline{x_2}, \ldots, \overline{x_n}) \tag{6}$$

In this reduced variable space, the limit state surface (g(X) = 0) separates safe and unsafe states, See Figure 11 and the reliability index (β) can be defined as the distance from the origin to the most probable failure or design point. This distance can be described as:

$$\beta = \min\sqrt{X^T X} \quad \text{for } X \in \Psi \tag{7}$$

Where:

X vector representing the set of reduced random variables;
Ψ failure region defined by (g(X) = 0).

The probability of failure P_f can then be calculated by integrating the failure region.

$$P_f = P\left[g(X) < 0\right] \tag{8}$$

Where the notation g<0 denotes the failure region.

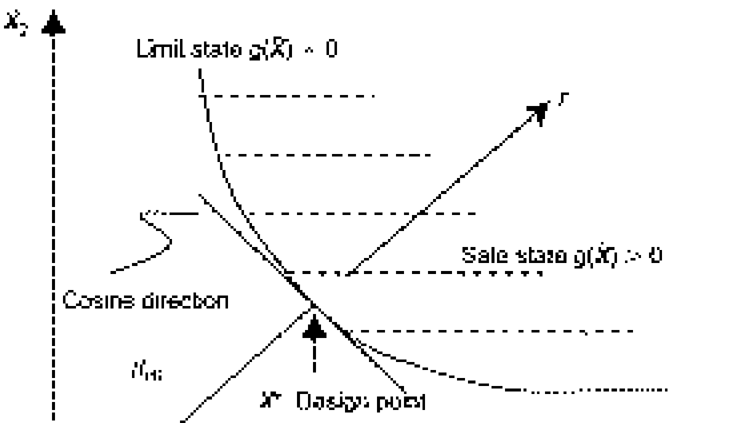

Figure 11 Definition of failure surface in polar coordinates

The design values assumed in the analysis are shown in Table 2. The variation in reliability index as the wetting front depth increased from 0.1m to 2m is shown in Figure 12. Comparing results of the reliability index with the deterministic analysis, it is clear that the reliability index decreases at a much slower rate as the wetting front develops than the decrease in sharp safety predicted by the FOS. This is a result of the relatively low variability of many of the input parameters. As a result we can see that at a wetting front depth of 1 m we have a reliability index of 2.75, which is substantially higher than the minimum β value of 2.2 required by most infrastructure owners.

Table 2 Summary table of probabilistic inputs

Parameter	Mean Value	Distribution
Cohesion (c')	0	–
Soil suction (u_a-u_w)	7 kPa	Lognormal
Internal Angle of Friction	34°	Normal
Rate of increase in shear strength due to matric suction Φ^b	24°	Normal
Slope angle	38°	–
Unit weight	17 kN/m^3	Lognormal
Wetting front depth	Varied from 0.1 m to 2 m	–

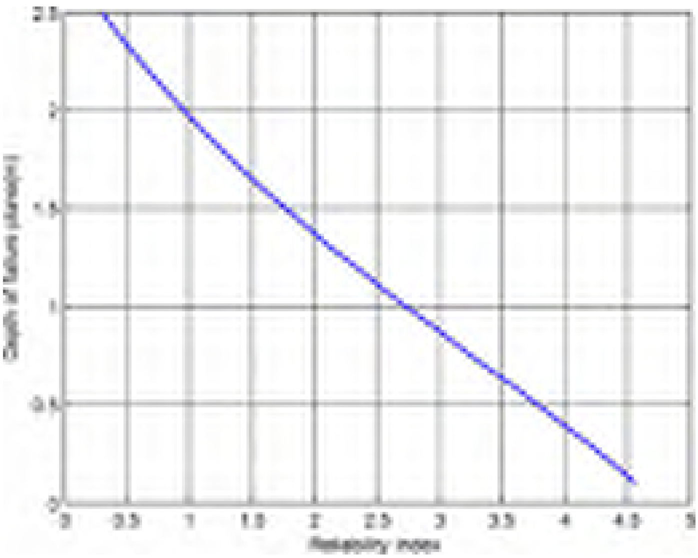

Figure 12 Results of probabilistic analysis

It is worth noting that the suction measurements shown in the histogram in Figure 10 are not truly random variables. Rather, the suction varies in response to applied rainfall. The next stage of the model development is to derive a method for predicting the wetting front depth during given rainfall events, and the variation of wetting front depth due to the variation of soil permeability. This will allow the designer to choose rational values of suction (or water content) in the slope for a given climate condition. It is expected that fragility curves will be developed to describe the slope response to a range of possible climate scenarios.

5 Conclusions

Infrastructure managers across Europe are facing challenges in managing aging infrastructure with reduced resources. Climate change effects are causing increased stress on many elements of infrastructure. This paper examined methods considered in the EU funded SMARTRAIL project of identifying where slope stability problems might occur along a transport network and how the stability analyses should be performed.

Simple geophysical techniques were shown to provide large quantities of data on the in-situ state of slopes. Whilst visual assessments will remain a central element of the management of slopes, it should not be the primary method of determining risk ranking through some hazard assessment framework. New developments in monitoring and assessment, being developed in current EU funded projects, include remote structural health monitoring using either embedded sensors or vehicle mounted methods. In addition the use of drones to collect data or to undertake rapid and safe video capture of data, to allow for innovative assessment of rock slope stability are being developed.

Probabilistic tools are extremely useful for extending the service life of existing infrastructure as they give a more accurate representation of performance. Deterministic approaches which assume constant values for parameters which vary temporally cannot provide an adequate description of the performance of an asset. A mathematically rigorous approach for analysing slopes is presented. The important soil parameters can be determined using simple laboratory tests. Consideration of the performance of a 150 year old railway slope showed that despite having very steep side slopes the embankments is relatively stable and a very extreme rainfall event would be required to trigger failure.

Acknowledgements

The work described herein is funded through the SMARTRAIL Project funded by the EU FP7 Grant agreement no: 285683. Thanks are due to Dr. Shane Donohue, to Irish Rail and colleagues at University College Dublin.

References

[1] Chowdury, R.N, Grivas, D.A. (1982), "Probabilistic model of progressive failure of slopes", Journal of Geotechnical Engineering Division, Vol. 108, No. 6, pp 803-819.

[2] Christian, J.T., Ladd, C.C., Baecher, G.B. (1994), "Reliability applied to slope stability analysis.", J. Geotech. Engrg., Vol 120, No. 12, pp 2180–2207.

[3] Directorate General for Mobility and Transport, (2013), The Performing Rail Infrastructure Manager, European Commission (2013)

[4] Donohue, S., Gavin, K., Tolooiyan, A. (2011), "Geophysical and geotechnical assessment of a railway embankment failure", Journal of Near Surface Geophysics. Vol 9, No.1, , pp 33-44, DOI: 10.3997/1873-0604.2010040

[5] Donohue, S., Gavin, K., Long, M., O'Connor, P. (2003), "Determination of the shear stiffness of Dublin boulder clay using geophysical techniques" Proceeding of the 13th European Conference on Soil Mechanics and Foundation Engineering, Vol 3, pp 515-520. Prague August (2003)

[6] Fredlund DG., Morgenstern NR., Widger RA., (1978), "The shear strength of unsaturated soils", Canadian Geotechnical Journal, 15(3): 313-321, 10.1139/t78-029

[7] Fourie, A.B., Owe, D.R., Blight, G.E., "The effect of infiltration on the stability of the slopes of a dry ash dump". 1999, Geotechnique, 49(1): 1-13.

[8] Gavin, K., Xue, J, (2009), "Use of a genetic algorithm to perform reliability analysis of unsaturated soil slopes" Geotechnique, Vol. 59, No. 6, 2009, pp 545-549. DOI 10.1680/geot.8.T.004

[9] Gan J. K. M., Fredlund, D. G., Rahardjo, H. (1988), "Determination of the shear strength parameters of an unsaturated soil using the direct shear test", Can. Geotech. J. 25, 500-510

[10] Hassan, A. M., T. F. Wolff (1999), "Search algorithm for minimum reliability index of earth slopes", Journal of Geotechnical and Geoenvironment Engineering. ASCE 125(4): 301-308.

[11] Jennings, P., Muldoon, P. (2003), "Assessment of Stability of Man-made Slopes in Glacial Till: Case Study of Railway Slopes, Southwest Irelandl", Proceeding of the 13th European Conference on Soil Mechanics and Foundation Engineering, Prague August (2003)

[12] Low, B. K. (1996), "Practical probabilistic approach using spreadsheet." Uncertainty in the geologic environment: From theory to practice, C. D. Shakelford, P. P. Nelson, and M. J. S. Roth, eds., ASCE, New York, pp 1284–1302.

[13] Low, B. K., and Tang, W. H. 1997) "Reliability analysis of reinforced embankments on soft ground." Canadian Geotechnical Journal, Vol 34, pp 672-685

[14] Rackwitz, R. and Fiessler, B. (1978), "Structural reliability under combined random load sequences", Computers and Structures, Volume 9, Issue 5, pp 489-494.

[15] Reale, C. (2015) The effect of climate change on the stability of steep slopes, field tests and modeling. PhD thesis in preparation, University College Dublin.

[16] Reale, C., Gavin, K. , O'Connor, A. (2012), "Laboratory experiment to measure soil matrix suction and examine the effect of infiltration", Proc. BCRI 2012, 6th-7th September 2012, Dublin.

[17] Xue, J. and Gavin, K. (2007), "Simultaneous determination of the critical slip surface and reliability index for slopes", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Volume 133, No.7, July 2007, pp 878-886

1 RAIL INFRASTRUCTURE PROJECTS DESIGN, CONSTRUCTION, MAINTENANCE AND MANAGEMENT

OPTIMISATION OF RAILWAY OPERATION BY APPLICATION OF KRONECKER ALGEBRA

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Abstract

Kronecker Algebra consists of Kronecker Product and Kronecker Sum. This mathematical model can be used to model systems consisting of a number of limited resources and several actors. In particular, it can be used to model railway systems with trains, track sections and their routes. In this paper we show several applications of Kronecker Algebra in the railway domain. We consider deadlock prevention, travel time calculation, and energy analysis. The integration of these three tasks within one single type of Kronecker-based analysis is rather simple and can be carried out very efficiently. Due to the fact that Kronecker Algebra operations can be easily parallelized, our implementation can take full advantages of today's multi-core computer architecture. In addition our implementation shows that adding constrains for connections or overtaking speeds up the calculation. In fact, a harder problem is easier to solve.

Keywords: Kronecker Algebra, railway systems, deadlock, travel time, energy analysis

1 Introduction

We present a graph-based method for analysis and optimization of railway networks on a fine-grained level. The routes of trains within this network are modelled by graphs. Each route consists of at least one track section and each track section may be part of several routes. We assume that at the same time only one single train occupies a track section. Our model employs semaphores [1] in the sense of computer science to guarantee that only one train enters a track section. The semaphores are also modelled by graphs. Each graph can be represented by its adjacency matrix. Simple matrix operations can be used to model concurrency and synchronization via semaphores. These matrix operations are known as Kronecker Sum and Kronecker Product and are part of the so called Kronecker Algebra. By applying these operations, we can get a matrix describing the whole railway system; in particular it contains all movements of the trains within this network. The whole theoretical background, including several examples of the applications of Kronecker Algebra, was published in previous papers [2], [3], and [4].This paper contains some advanced examples.

2 Example

Figure 1 shows a railway network with 9 track sections. Our example contains four trains which have the following routes:
· Train 1 (T1): 1-5-7 (p5, v1, p7, v5, v7)
· Train 2 (T2): 2-4-5-6-9 (p4, v2, p5, v4, p6, v5, p9, v6, v9)
· Train 3 (T3): 3-4-5-6-8 (p4, v3, p5, v4, p6, v5, p8, v6, v8)
· Train 4 (T4): 8-6-5-1 (p6, v8, p5, v6, p1, v5, v1)

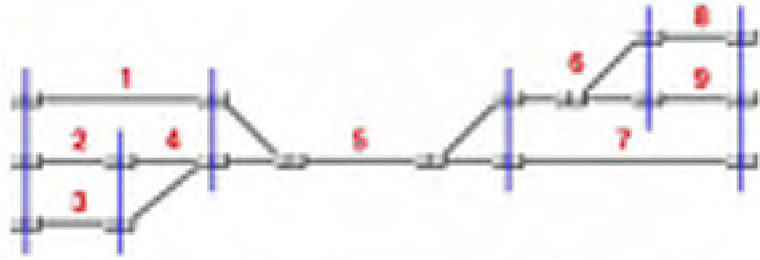

Figure 1 Railway system

Figure 2 Resulting graph (start node is the most left node)

The resulting graph of this example is illustrated in Figure 2. It contains 802 nodes, including yellow, red and green nodes, which have the following meaning:

- Red nodes denote deadlocks. These nodes are eliminated before the following calculations because we are interested in avoiding deadlocks.
- Green nodes denote safe states. A state is safe if all trains can perform their actions without having to take into account the moves of the other trains in the system, provided that the track section which they are to enter is not occupied by another train. If a track section is occupied by another train, the movement of the train wanting to enter may be delayed (blocked) but no deadlock can occur.
- From yellow nodes both red and green nodes can be reached.

Due to the size of the resulting graph, the node-ids and the edge labels are removed for a clear illustration. In general, each edge is labelled by the actions of the trains (cf. [2], [3], and [4]). Our implementation of Kronecker Algebra is very efficient in space and time, because we use a lazy implementation, where only the reachable parts of the matrix is calculated (cf. [3]). The calculation of the resulting graph of our example takes about 30 μs. As we are interested in the calculation of the worst-case travel time of each train within the railway network we define the following travel times for the trains on the track sections (Table 1). The columns labelled with "1" to "9" denote the track section.

Table 1 Travel time values

Train	1	2	3	4	5	6	7	8	9
T1	5				5		5		
T2		2		3	5	3			2
T3			2	3	5	3		2	
T4	3				3	2		1	

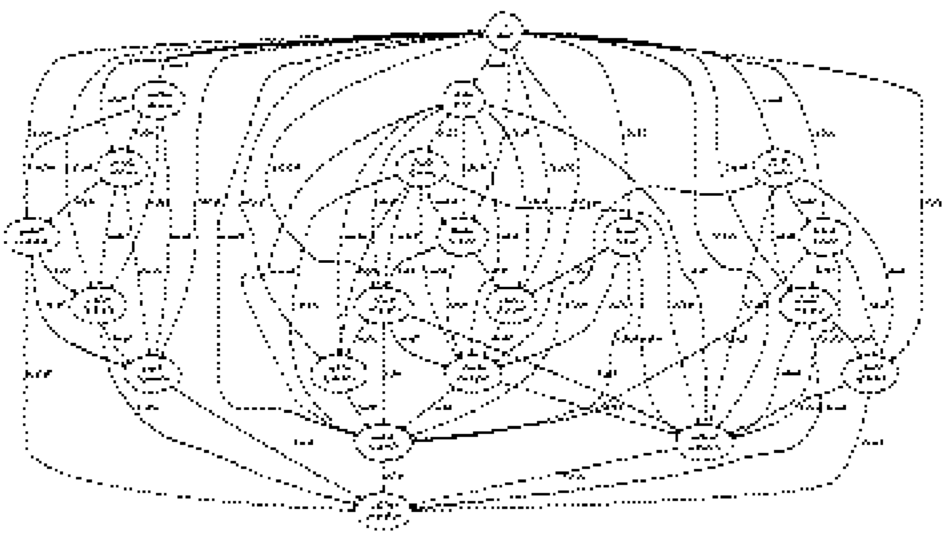

Figure 3 Reduced graph

Assuming that each train is independent from the others, the complete travel time is the sum of the travel times of all track section of the train's route. As there exist several common used track sections, there is an influence of the travel time of the trains, which use the track

sections. Further information about the synchronization between trains and the effect on their travel time can be found in [3] and [4]. The resulting travel time of the trains can be found in the second and third column of Table 2. The graph can be reduced to its relevant synchronizing nodes. Due to space restriction, the reduction algorithm is not explained here in detail. The reduced graph of our example, including the travel time values for the four trains at the nodes and along the path between them can be found in Figure 3.
In our example we used track sections as commonly used resources. Our model can be extended to use other shared resources, for example available energy (cf. [5], [6], and [7]). The available energy is quantised into standardised packages, e.g. 1 MWh. We can model a power station or substation capable of producing e.g. 20 MW by using a counting semaphore of size 20 (cf. [5], [6], and [7]).
Now we extend our model. For each train it is known a priori, how much energy is needed on a track section. This amount of energy is reserved before entering the track section and released afterwards. Modelling available energy and required energy in this way ensures that a train needing more energy than can be delivered by the power station is blocked. In particular, it will stop its journey and can continue when enough energy is available (e.g. because another train has released its energy needs or the station can provide a higher number of energy packages). Table 2 shows the energy demand of the four trains within our railway system and their needs of energy for each track section.

Table 2 Energy demand

Train	1	2	3	4	5	6	7	8	9
T1	2				2		2		
T2		1		1	2	1			1
T3			1	1	2	1		1	
T4	2				2	1		1	

It is obvious, that the maximum amount of energy at a given time is eight. If the station can deliver at least eight energy units, there is no effect on the travel time of the four trains (Table 3, fourth column). The size of the graph increases to 6443 nodes, because an additional counting semaphore of size 8 is added to our calculations. The influence on the travel time can be found in columns five, six, and seven of Table 3, if there are only six, four, or two energy units available, respectively. Obviously, the travel time increases because some of the trains will have to stop and wait until enough energy is available to continue their travel. The size of the resulting graph decreases, because we specify the problem in more detail and thus the number of possible train movements decreases which has an effect on the size of the graph. By application of Kronecker Algebra, a harder problem is easier to solve. The last column of Table 3 shows the result, if only one energy unit is available. As a consequence, no train can finish its journey, because each of them will need at least two available energy units for particular track sections. The resulting graphs for two and one available energy units are given in Figures 4 and 5.

Table 3 Resulting travel time

Train	Travel Time	Worst cast travel time	Travel time E=8	Travel time E=6	Travel time E=4	Travel time E=2	Travel time E=1
T1	15	25	25	30	30	30	∞
T2	13	36	36	41	49	52	∞
T3	15	38	38	43	51	54	∞
T4	9	28	28	33	39	39	∞
No. of nodes	802	802	6443	6053	3147	418	11

Figure 4 Resulting graph (two energy units)

Figure 5 Resulting graph (one energy unit)

In Figure 5, it can be seen that each train can perform its first action, but then it deadlocks, due to the lack of available energy. In our examples we have used integer values for travel time values. Extensions of our model, e.g. with decimal values or taking braking and acceleration time into account in case of blocking are possible, but not discussed here.

3 Conclusion

We have presented a practical example for the application of Kronecker Algebra, where we model the movements of trains within a railway system and the access on a shared track section. Afterwards the travel time of each train can be calculated. We extended the example by the modelling of energy units, produced by a power station and analyzed the results, based on the available amount of energy. Our approach can be used to model complex railway systems including aspects of being deadlock-free, being conflict-free, and being minimal in terms of energy demand. The theoretical background of Kronecker Algebra can be found in some preliminary papers ([2], [3], [4]. [7])

Acknowledgements

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References

[1] Dijkstra, E. W.: "Over Seinpalen," 1965.

[2] Mittermayr, R., Blieberger, J.: "Timing Analysis of Concurrent Programs," Proc. 12th International Workshop on Worst-Case Execution Time Analysis, pp. 59-68, 2012.

[3] Mittermayr, R., Blieberger, J., Schöbel, A.: "Kronecker Algebra based Deadlock Analysis for Railway Systems," PROMET-TRAFFIC & TRANSPORTATION, pp. 359-369, 2012.

[4] Volcic, M., Blieberger, J., Schöbel, A.: "Kronecker Algebra based Travel Time Analysis for Railway Systems," FORMS/FORMAT 2012 – 9th Symposium on Formal Methods for Automation and Safety in Railway and Automotive Systems, pp. 273-281, December 2012.

[5] Volcic, M., Blieberger, J., Schöbel, A.: "Kronecker algebra and its broad applications in railway systems," EURO-ŽEL 2013: Recent Challenges for European Railways, pp. 275-282, June 2013.

[6] Volcic, M., Blieberger, J., Schöbel, A.: "Kronecker algebra as a frame for optimisation of railway operation," 21st International Scientific Conference – TRANSPORT 2013; Mechanics Transport Communications, vol. 11, no. 3, pp. 57-63, October 2013.

[7] Volcic, M., Blieberger, J., Schöbel, A.: "Kronecker algebra based modelling of railway operation,"MT-ITS 2013, pp. 345-356, December 2013.

[8] Mittermayr, R., Blieberger, J.: "Shared Memory Concurrent System Verification using Kronecker Algebra," Automation Systems Group, Technische Universität Wien, Wien, Österreich, 2011.

THE STUDY ON GROUND BEHAVIOR BY STEEL PIPE JACKING BASED ON A FULL-SCALE TEST

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Abstract

Though construction of underground structure passing beneath the railroad or road has been increasingly carried out using non-open cut excavation method, the study on ground behavior and settlement/heaving of trackbed while jacking the steel pipe has yet to be put on track. This study is intended to evaluate the displacement depending on top soil depth in a bid to identify the ground behavior caused by pipe jacking based on a full scale test. As a result of the test, heaving reached the peak when the top soil was shallow which then tended to decrease in line with increasing top soil depth.

Keywords: full scale test, underground crossing structure, non-open cut method

1 Introduction

Construction method to pass beneath the railroad in operation has been commonly applied to various sites. Non-open cut excavation method, among others, is designed to keep putting the steel pipe or square pipe into the ground continuously while excavating and hauling the soil [1] When the pipe is penetrated into the ground, heaving or settlement usually occurs at upper level of the ground. Heaving occurs when jacking force is greater than the effect by face releasing while settlement occurs when the effect by face releasing is greater than jacking force. When settlement following the heaving occurs after encountering the obstacle, soil erosion by running groundwater or face collapse occurs frequently [2].
According to Yoo, Chung-sik (2009), the factor which directly affects ground surface settlement, among other factors such as ground level, consolidation degree and ground elastic modulus, is top soil depth [3] Eum, Kee-young et al (2010) conducted the model test of underground structure with various top soil depth and compaction degree [5].
According to “the study on construction of underground structure passing beneath the high speed railroad and stability evaluation” non-open cut method to pass through beneath the structure since 1999 totaled 56, indicating the increase recently [6] but except such cases, the study on ground behavior and settlement/heaving of trackbed due to steel pipe jacking has yet to be made in earnest.
Thus in this study, a full-scale lab test was planned to comprehensively analyze the behavior beneath the trackbed while jacking the steel pipe into the ground. Basic property examination was carried out and a full-scale ground was formed by banking the soil gradually. And ground behavior and heaving/settlement when jacking the steel pipe used at the construction site were analyzed as well as the effect depending on top soil dept.

2 A full-scale test

2.1 Test device

Figure 1 shows the general trackbed test device, which was designed to reproduce actual filed condition and rail load at lab so as to solve the problem that might occur at the site. This test device comprises the chamber, loading device (MTS) and load reaction table reacting against the load. Chamber used for the test was 5.0m wide, 22.0 m long and 3,0 m deep and has the access ramp for the equipment.

Figure 1 General trackbed tester

Figure 2 Model chamber

2.2 Engineering characteristics of test ground

Trackbed was formed with natural soil with high bearing capacity, low compressibility with no mud pumping. It's also stable against vibration and rainfall and has the strength to bear the train load. To identify the physical & dynamic properties of soil used as trackbed material, following lab test was conducted.

As seen in Table 1, grain size distribution test of soil (KS F 2302) was conducted, and as a result, a grain-size distribution curve as shown in Figure 3 was obtained and according to grain size distribution test and unified soil classification system based on Atterberg limit test in Table 2, sample soil was determined as SC. Maximum dry density and optimal water content were obtained from water content – dry density curve as seen in Figure 4 which was then used for compaction evaluation of trackbed for full-scale test. A direct shearing test (KS F 2343) was conducted. As a result, adhesion C was 20.6 kPa and internal frictional angle Ø was 33°. Figure 5 shows shear stress change depending on horizontal strain when vertical load was 50 kPa, 100 kPa and 150 kPa and Figure 6 shows Mohr-Coulomb failure envelope.

Table 1 Result of Size Distribution

Sieve	#4	#10	#16	#40	#60	#100	#200
%	84.6	67.7	56.4	39.8	31.5	12.8	9.15

Table 2 Result of C_u and C_c

	#200 (%)	D_{10}	C_u	C_c
No.1	12.9	0.12	12.5	0.32

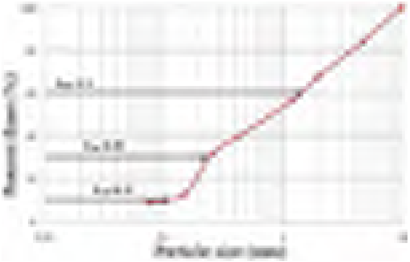


Figure 3 Size distribution curve

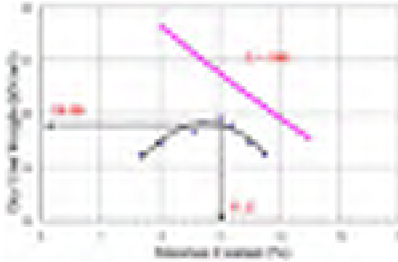


Figure 4 Compaction curve

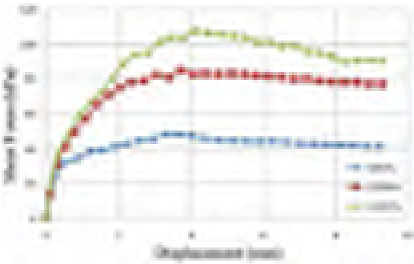


Figure 5 Shear stress curve

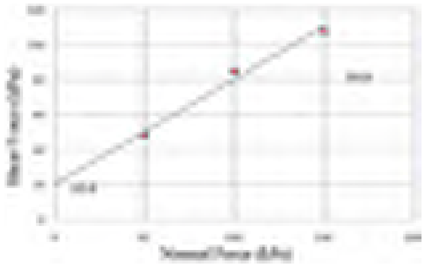
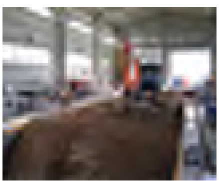


Figure 6 Failure envelop

2.3 Test condition and method

Pipe jacking method includes non-displacement method and auger screw method. When the leading pipe is jacked into the ground, following pipe is connected and advanced into the ground. Soil and advanced distance are dependent on type of leading pipe in this method but 250~600 mm is mostly used. Ground was formed according to the work sequence of corrugated plate and the test was conducted in following sequence to ensure the test accuracy is maintained.

a) Install reaction wall



b) Fill up

c) Compact ground

d) Measure

e) Install LVDT

f) Install jacking device

g) Press pipe

h) Dig in pipe

Figure 7 Pipe construction process

According to Kim, Young-ha et al (2013) and as a result of analysis of behavior of the structure crossing under the ground through the scaled model test, top soil depth was the most influential factor on ground behavior. Thus in this full-scale test, top soil depth was determined as test condition and top soil depth was set three cases as 60 cm, 90 cm and 120 cm as shown in Table 3 so that the top soil depth – pipe diameter ratio would be 1.0, 1.5 and 2.0

Table 3 Experiment case

	Soil depth	H/D
Case 1	60 cm	1.0
Case 2	90 cm	1.5
Case 3	120 cm	2.0

3 Ground surface displacement analysis

3.1 Measuring the point where maximum heaving occurred by steel pipe jacking

Steel pipe jacking in a full-scale test was carried out in a way of manual excavation after pressing the pipe into the ground. After putting the pipe 50 cm, manual excavation was followed and total 10 to 15 cycles were repeated depending on top soil depth. Figure 8 shows the symbols of measuring point.

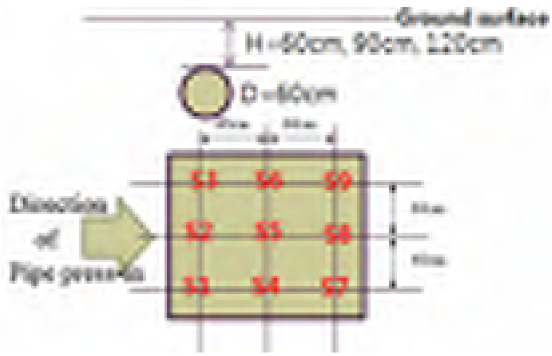


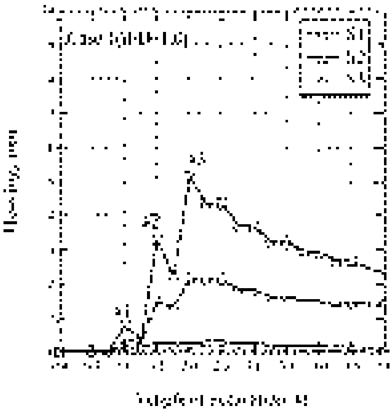
Figure 8 Measurement point of surface

Figure 9 shows ground surface displacement caused by pipe jacking at H/D=1.0. At the test with most shallow top soil, measuring point when a single pipe is jacked was ay fore-end S1, S2 & S3. As seen in Fig, the greatest deformation was occurred at penetration point and maximum heaving about 5.2 mm was occurred at 2m point, which was gradually reduced as pipe passed and was reduced to 2.4 mm or by 60% at final excavation point at 5 m. Heaving was reduced during excavation after jacking, which indicated settlement occurred due to face releasing and the behavior by pipe jacking such as heaving and settlement was indirectly verified to have varied by the influence around the face. Such behavior seemed to be same as the result from scale model test and steel pipe face at the site was found to be very important barometer to ground surface behavior. Given the heaving is dependent on jacking at top soil depth (H/D=1.0), it shall be considered in planning the construction.
Figure 10 shows the ground surface displacement pattern measured at the fore-end and center when top soil depth H/D=1.5. As indicated in Figure, when top soil depth was 1.5 times or more the diameter, heaving behavior was relatively smaller. Displacement even at the fore-end was less than 2 mm and when comparing to the case of top soil depth H/D=1.0, reduction in heaving was more than 3 times. Viewing such result, ground surface displacement was very insignificant when top soil depth is H/D=1.5 or more.

Figure 11 shows the surface displacement pattern measured at the fore-end and center with top soil depth H/D=2. Under H/D=2, heaving while pipe jacking was very insignificant, unlike previous test conditions while settlement behavior was dominant, which was attributable to greater settlement behavior than heaving.

Increasing top soil weight constrained the heaving behavior and after pipe passed the measuring point, settlement occurred by soil weight. That is, heaving behavior was constrained by dead weight when the pipe reached the measuring point and settlement occurred due to rearrangement of loosened ground after pipe passed the measuring point. Thus, when top soil depth (H/D) is more than twice, settlement is dominant, whereas heaving is dominant when top soil depth is smaller. In the event of penetrating the multiple pipes at top level, the effect of secondary or additional pipes is expected depending on loosened territory by diameter of initial pipe or diameter ratio, but the effect was monitored to be insignificant. But given the heaving and settlement behavior would possibly be occurred simultaneously when jacking the pipe due to ground material and density and face conditions or other various factors, face releasing territory shall be enlarged at early stage of jacking when top soil depth is shallow to deal with heaving behavior and when top soil is deep, settlement is dominant and thus the measure to reduce the gap caused by rearrangement of loosened soil as well as ground less shall be taken.

a)



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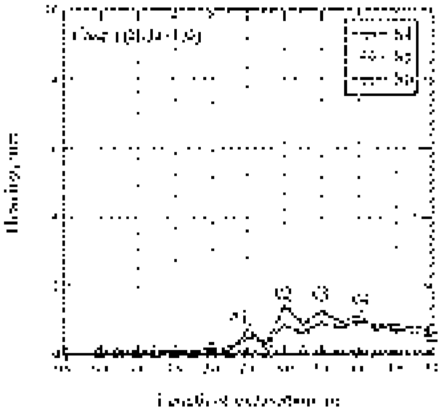
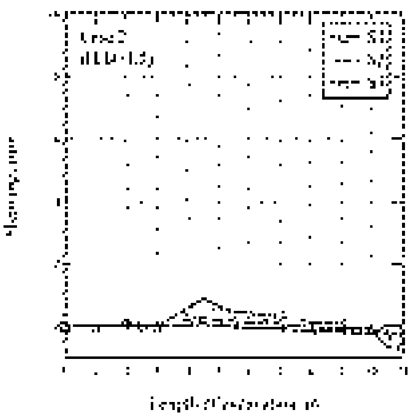


Figure 9 Displacement by stage in case1 (H/D=1.0)

a)



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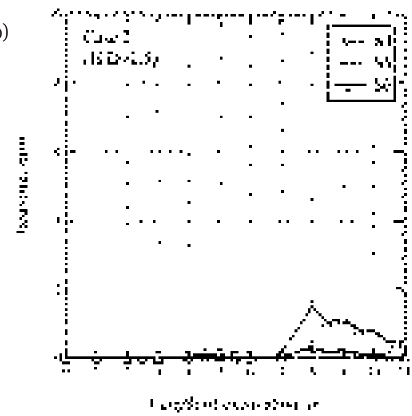

Figure 10 Displacement by stage in case1 (H/D=1.5)

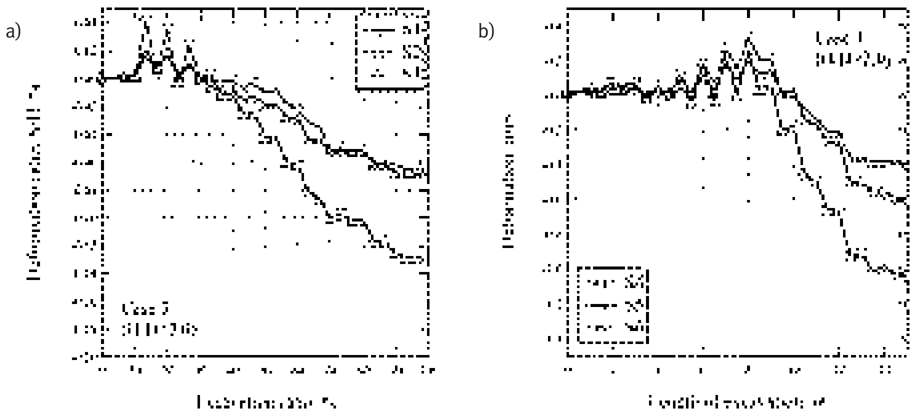

Figure 11 Displacement by stage in case1 (H/D=2.0)

4 Conclusion

The study on ground behavior and settlement/heaving of trackbed while jacking the steel pipe for underground structure using non-open cut excavation method has yet to developed in earnest. In this study, ground surface behavior was monitored through a full-scale test to review the ground behavior caused by pipe jacking and the conclusion was made as follows.

1 As a result of measuring ground surface deformation depending on top soil depth under same ground conditions and steel jacking condition, heaving and settlement varied significantly depending on top soil depth. In case of the test with most shallow top soil H/D=1.0, maximum 5.2 mm heaving occurred when pipe was penetrated 2 m, which was then gradually reduced to 2.4 mm or by 60% when pipe reached the final excavation point. Heaving was reduced during excavation after jacking, which indicated settlement was occurred due to face releasing and the behavior by pipe jacking such as heaving and settlement was indirectly verified to have been varied by the conditions around the face.
2 Heaving behavior was very insignificant when top soil depth was H/D=1.5 or more. Displacement even at the fore-end was less than 2mm and when comparing to the case of top soil depth H/D=1.0, reduction in heaving was more than 3 times. Viewing such result, ground surface displacement was very insignificant when top soil depth was H/D=1.5 or more.
3 Settlement behavior became dominant over heaving as top soil became deeper and heaving and settlement were greatly affected by the face conditions. When top soil depth was more than 1.5 (H/D), effect by pipe diameter was very insignificant with very small deformation rate less than 0.2%. Thus, if top soil depth ratio is more than 1.5 times the pipe diameter, effect by top soil depth or pipe diameter would be insignificant.

Acknowledgement

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References

[1] Korea Railroad Research Institute: A Study on Underground Railroad Crossing Method Selection Criteria, 1999

[2] Eric Leca, Animateur,Barry New, General Reporter: Settlements induced by tunneling in Soft Ground, Tunneling and Underground Space Technology 22, pp. 119-149, 2007

[3] Yoo Chung-sik, Kim Ju-mi, Kim Seon-bin, Jeong Hye-yeong: GIS-ANN Based Urban Tunnel Design/ Construction Risk Assessment, Journal of Korea Society of Civil Engineers, Vol. 26, No. 1 C, pp. 63-72, 2006

[4] Yuk Jeong-hun, Kim Nak-yeong: Behavioral Characteristics according to Height of Soil Cover of Corrugated Steel Pipe Underground Structures, Journal of Korea Geotechnical Society, Vol. 5, No. 1, pp. 65-73, 2004

[5] Eum Ki-young, Choi Chan-yong, Cheon Jeongyeon: Analysis of Scale Model Uplift and Settlement Experiments according to Shape of Pipes in Non-Excavation Propulsion Method Journal of Korea Geosynthetics Society, Vol. 9, No. 4, pp. 9-15, 2010

[6] Korea Railroad Research Institute: Research on Safety Evaluation and Establishment of Standards for Method of High- Speed Rail Underground Crossing Structures, 2010

[7] Young-Ha Kim, Ki-Young Eum and Jae-Wang Kim: A Study on Behavioral Characteristics of Track Roadbed according to Steel Pipe Press-in Excavation during Construction of Underground Railway Crossing, International Journal of Railway, Vol.6, No.2, pp. 69-77, 2013

DEVELOPMENT OF A HEATING SYSTEM FOR HOLLOW SLEEPERS CONTAINING POINTS POSITIONING SYSTEMS

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Abstract

In winter, malfunctions of points on high speed routes can occur due to driving snow and ice. Existing heating systems, used to increase the availability of points positioning systems contained in hollow sleepers, cannot guarantee their operation under harsh environmental conditions. An optimized heating system is drafted using computational and experimental methods. The impacts of various designs of heating systems on the temperature profile at a positioning system are assessed using the thermal network method. A thermal network of the hollow sleeper and the positioning system equipped with an existing heating system is compiled and verified with experiments. The experimental verification is required to minimize the uncertainty of computed temperature profiles resulting from the uncertainty of flow and material parameters in the thermal network model. The efficiency of differently designed heating systems is calculated from computed temperature profiles of the points positioning system. A design with allocated heating elements is investigated experimentally in order to verify the computational results of that design. The temperature rise achieved with the optimized heating system is significantly higher than the one achieved with the original system, while the admissible temperatures are not exceeded.

Keywords: heating system, points, hollow sleeper, thermal network method, heat transfer, temperature rise

1 Introduction

Reliable carriage by rail requires a high availability of the infrastructure. In winter, malfunctions of points can occur due to snow and ice. On high speed routes, driving snow can ingress into hollow sleepers containing the points positioning system and impair its mobility. The preferred solution to increase the availability of points and in particular the availability of the positioning system in hollow sleepers is to apply heating systems. At all times in operation and especially under moderate environmental conditions, the maximum admissible temperature of the positioning system, which is determined by the hydraulic system, must not be exceeded at any spot in the points positioning system. Hence, the power of the heating system has to be limited in order to fulfil this task. Existing heating systems therefore cannot guarantee the operation of the positioning system under harsh environmental conditions like environmental temperatures as low as 20 °C in winter. Their capability of removing snow and ice does not suffice. Hence, an optimized heating system is required, which inherently does not exceed the admissible temperatures and provides an increased capability of removing snow and ice from the positioning system.

2 Thermal network method

The thermal network method [1] enables the investigation of temperature rise and temperature distribution within complex arrangements like points positioning systems contained in hollow sleepers. Within a thermal network, the heat transfer processes are simulated with the help of heat sources, temperature sources, thermal resistances and thermal capacities [2]. This method of computation is based on the analogy of the electric and the thermal flow field. The temperature ϑ of the thermal field is analogue to the electric potential ϕ of the electric field and the heat flow P is analogue to the electrical current I (Table 1).

Table 1 Analogy relations between the electric and thermal field

Electric field			**Thermal field**		
Magnitude	Symbol	Unit	Magnitude	Symbol	Unit
Potential	φ	V	Temperature	ϑ	°C
Potential difference voltage	$\Delta\varphi$ U	V	Temperature difference	$\Delta\vartheta$	K
Current	I	A	Heat flow	P	W
Resistance	R_{el}	$V\,A^{-1}$	Resistance	R_{th}	$K\,W^{-1}$

The heat flow is fed into the thermal network by heat sources, that e. g. mimic a section of the flat heating inside a hollow sleeper. The heat flow P is calculated with

$$P = I^2 R_{el} \tag{1}$$

The heat transfer processes in thermal networks are simulated by thermal resis-tances, defined as

$$R_{th} = \Delta\vartheta\, P^{-1} \tag{2}$$

The heat transfer processes of conduction, convection, thermal radiation and naturally driven volumetric flow occur in the hollow sleeper. The conductively transferred heat flow is described by FOURIER`s law of heat conduction [3], [4]:

$$P_d = -\lambda\, \vec{A}\, \mathrm{grad}\vartheta \tag{3}$$

λ is the thermal conductivity, A is the area of the heat flow and gradϑ is the gradient of the temperature field. The convective heat transfer is given by NEWTON'S law, with the convection coefficient α_{co}, the surface area A_{co} and the temperature difference $\Delta\vartheta$ between the surface and the fluid:

$$P_{co} = \alpha_{co} A_{co}\, \Delta\vartheta \tag{4}$$

The heat transfer coefficient α_{co} contains the physical flow processes and is determined by the similarity theory with the Nusselt number Nu, the Rayleigh number Ra and the characteristic length l_w [2]:

$$\alpha_{co} = Nu\, \lambda_{med}\, l_w^{-1} = c_1\, Ra^{n1}\, \lambda_{med}\, l_w^{-1} \tag{5}$$

The parameters c_1 and n_1 are a function of the flow geometry and are given for basic assemblies in e. g. [2]. The radiation heat flow P_{rad} between two bodies i and j is given by the STEFAN BOLTZMANN Law:

$$P_{rad} = \varepsilon_{i,j}\, C_s\, A_i \left[T_i^4 - T_j^4\right] \tag{6}$$

It contains the resulting emissivity $\varepsilon_{i,j}$, the radiation coefficient C_s of the black body, the radiating surface area A_i and the absolute temperatures T_i and T_j.

$$\varepsilon_{i,j} = f(\varepsilon_i, \varepsilon_j, A_i, A_i, F_{i,j}) \tag{7}$$

The resulting emissivity $\varepsilon_{i,j}$ is a function of the emissivity ε the surfaces areas A of the radiating bodies and the view factor $F_{i,j}$ [3], [4], [5]. The heat transfer P_V via volumetric flow is a function of the specific heat capacity c_p, the fluid density δ_0, the volumetric flow rate $\dot{V}$and the temperature difference $\vartheta_i - \vartheta_j$ between the entering and exiting fluid.

$$P_V = c_p\, \delta_0\, \dot{V} \left(\vartheta_i - \vartheta_j\right) \tag{8}$$

The volumetric flow rate $\dot{V}$is a function of the buoyancy height h between the inlet and outlet apertures, the coefficient of volumetric thermal expansion β_0 and the flow resistances [2], [4]. The determination of flow parameters is generally not trivial and subject to uncertainty. Experimental verification is hence required.

3 Thermal network of positioning system in hollow sleeper

In this chapter the steps of modelling a points positioning system with the thermal network method as well as the compilation, structure, parameters and experimental verification of the model are described.

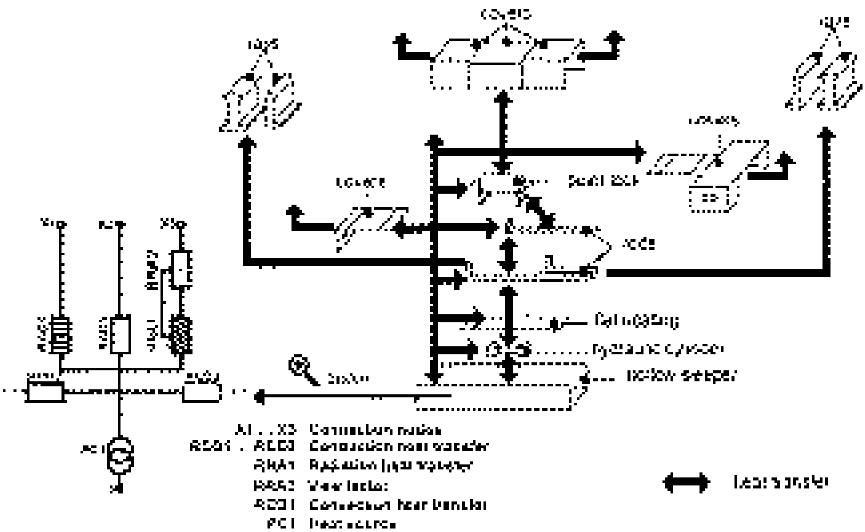

Figure 1 Thermal network model of the points positioning system

3.1 Classification of assemblies

The regarded points positioning system is a complex group of altogether 17 assemblies made of several materials with differing thermal material properties and surface qualities. The assemblies are: 9 different covers, 3 different rods, the hydraulic system, the rails, the point lock, the hollow sleeper and the flat heating. The thermal network model is compiled for each assembly separately and the resulting assembly models interact via conduction, convection and radiation heat transfer. Each assembly is modelled with several nodes (Figure 1), which interact with each other via conduction and with the other assemblies and the environment via convection and radiation. The radiative interaction between surfaces of several assemblies is a function of the geometry, temperature distribution and surface quality and it is described using view factors [4], [5]. The number of required nodes per assembly results from each assembly's dimension and its spatial constant of temperature distribution [2]. The heating system is modelled with spatially distributed power sources. The model requires several input parameters besides its structure to be defined. Input parameters are geometric (cross sections A, surface areas A, distances and view factors F_{ij}), material and surface properties [6] (thermal conductivity λ, emissivity ε) and flow parameters [2].

3.2 Verification

The material and surface properties depend on the chemical composition of the used steel alloys and their manufacturing procedure. An experimental verifica-tion of the thermal network model is necessary to limit the uncertainty of the computational results caused by the uncertainty of material and surface properties. The cover of the hollow sleeper exhibits several apertures, which enable a naturally driven volumetric flow between the inner air of the hollow sleeper and the environment. Flow parameters describing the volumetric flow and the convective heat transfer are given for basic assemblies [2] but need to be verified experimentally for complex assemblies. The verification process is performed in two steps in order to separately investigate convection flow parameters together with material and surface properties on the one hand and volumetric flow parameters on the other hand. The flat heating is operated with three different powers which are normalized with the power of the original heating system. The apertures in the cover of the hollow sleeper are closed and proofed in the experiments of the first step. A volumetric flow from the inside of the hollow sleeper to the environment cannot occur in this case (Figure 2).

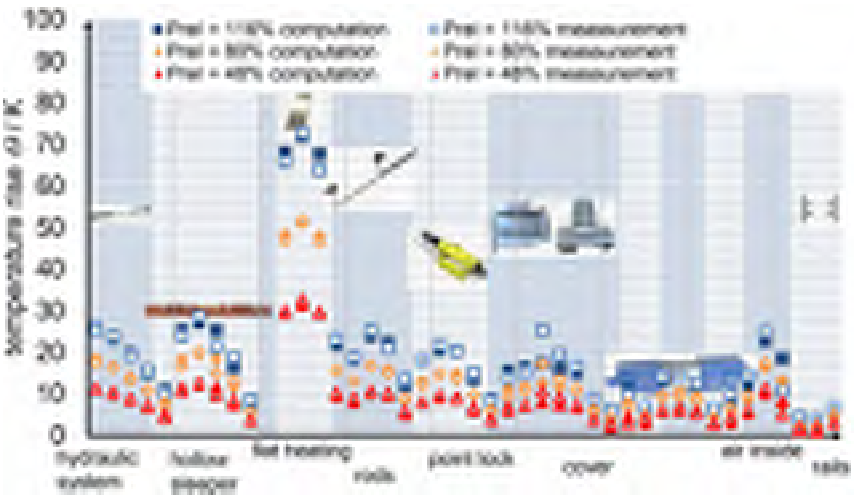

Figure 2 Experimental verification of thermal network with proofed apertures

In the experiments of the second step, the apertures are open and a volumetric flow from the inside of the hollow sleeper to the environment can establish. Its parameters are derived from the experimental results and complete the thermal network model of the entire positioning system contained in the hollow sleeper (Figure 3).

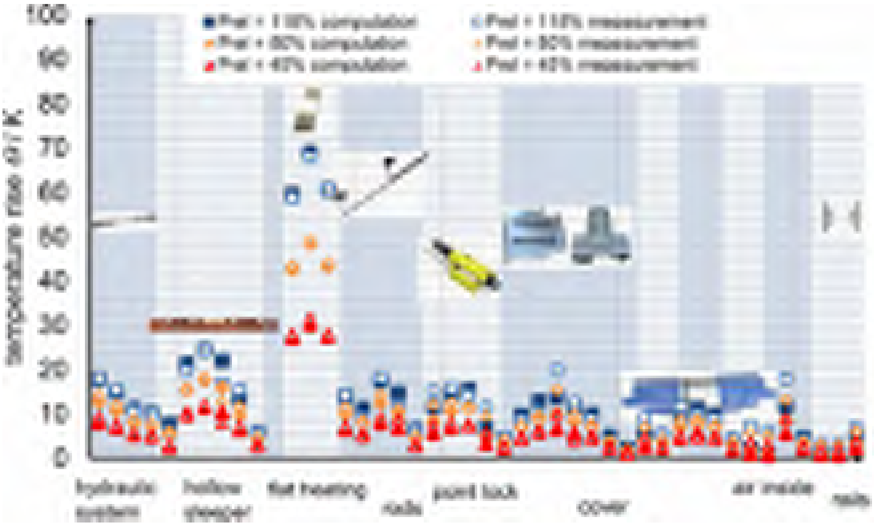

Figure 3 Experimental verification of thermal network with open apertures

4 Optimized designs of heating systems

As a result of the temperature rise tests (Figure 3) an uneven temperature distri-bution is evident along the hydraulic system, the hollow sleeper, the rods, the covers, the point lock and the flat heating. While the admissible temperature rise is exceeded at the flat heating, the temperature rise is less than 20 K in most of the positions. Therefore, high availability at winter conditions with outside temperatures as low as -20 °C cannot be ensured. By optimizing the heating system design, the temperature distribution in the entire hollow sleeper shall be homogenized. Increasing the surface area of the flat heating would obviously increase the heat transfer to the positioning system while reducing the surface temperature of the flat heating. Nevertheless, a flat heating with increased dimensions cannot be mounted in the hollow sleeper, as only limited room is available. Instead, the radiation heat transfer from the flat heating could be intensified by varnishing the surfaces of the flat heating, the gears and the inner surfaces of the hollow sleeper. Varnished surfaces have a higher emissivity of thermal radiation than blank metal surfaces [2], [6]. The usage of several allocated heating elements could homogenize the temperature distribution as well as directed heat transfer via a radiant heater combined with varnished surfaces. Alternatively, warm air can be fanned into the hollow sleeper. The impact of the introduced design alterations on the temperature distribution is assessed computationally (Figure 4) using the compiled thermal network model. The computational model considers an environmental temperature of 5 °C, which is the highest value to be expected in winter condition. Solar radiation is also taken into account for assessing the admissible power of the heating system.

Table 2 Parameters of the computed temperature rise distribution

Design	Mean value of temp. rise / K	Coefficient of variation
Original design	25.3	0.58
Varnished surfaces	27.4	0.54
Allocated heating elements	33.9	0.51
Flat + radiant heater	28.1	0.51
Warm air fan	19.7	0.47

The efficiency of the heating system designs is measured by the mean value of the temperature rise, while its homogeneity is measured by its coefficient of variation (Table 2). Only the temperatures of the hydraulic system, the hollow sleeper, the heating elements, the rods and the point lock are taken into account. The cover is not likely to impair the positioning systems mobility and the rails are separately heated which is not considered in the present investigation.

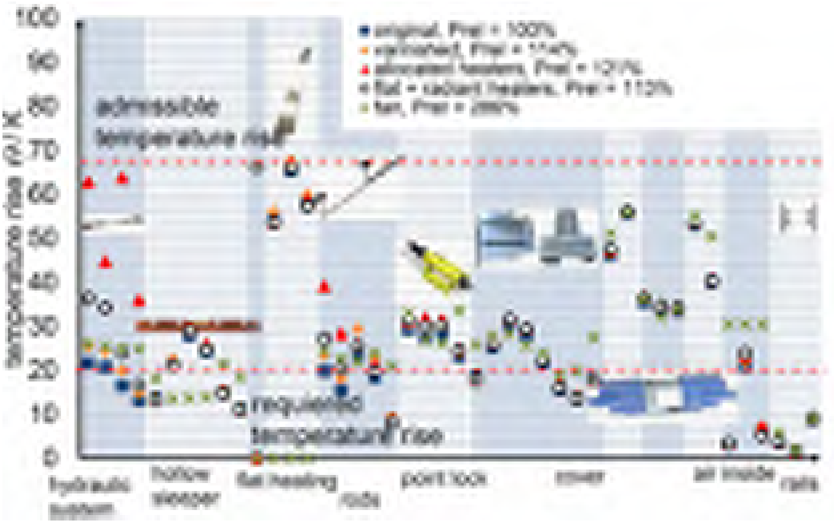

Figure 4 Computed temperature distribution for various designs of the heating system

The computational results only indicate a moderate effect of varnished surfaces on the temperature distribution within the positioning system. Fanning warm air into the hollow sleeper leads to a relatively homogenous temperature distribution but requires excessive power, because most of the heat is carried away by the warm air. Radiant heaters and allocated heating elements result in higher temperature rise and more homogenous temperature profiles than the original configuration.

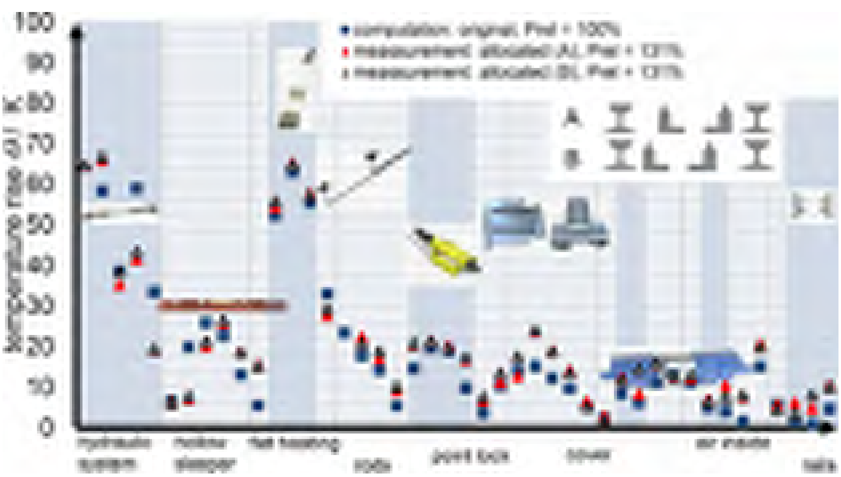

Figure 5 Experimental temperature rise with several allocated heating elements

The increase of the temperature profile homogeneity and temperature rise due to allocated heating elements is approved by experimental investigations (Figure 5). The temperature profile hardly depends on the position of the switch rails. The impact of the heating system optimization therfore does not depend on the switch rail position either. The power consumption of the optimized heating system with allocated heating elements is P_{rel} = 121 % and slightly higher compared to the original configuration. An increased effictivity of the optimized heating system can strictly be approved only under the conditions of investigation (no wind, no percipitation, environmental temperatur approximately 20 °C) and cannot be reasonned directly for realistic operational conditions. This is due to the impact of the environmental conditions, that has not been assessed experimentally. Wind increases the heat transfer to the environment and percipitation lowers the temperatures at the hollow sleeper. Therefore, a temperature rise of 20 K under laboratory conditions is only a necessary but not a sufficient condition for operation at environmental temperatures as cold as -20 °C. Nevertheless, the increase of mean temperature rise and the homogeneity of the temperature profile at least indicates a better availabillity of the points positioning system in winter conditions. It is not admissible to increase the power of the heating system in order to further increase its effectivity, because the admissible temperatures may be exceeded. Either higher admissible temperatures or a controll system could create the possibility for a further increase in heating power.

5 Conclusions

In order to assess the impact of various designs of heating systems on the tem-perature distribution at a points positioning system contained in a hollow sleeper, a thermal network model has been established and verified experimentally. Various modified designs of an existing heating system have been investigated by computational and also experimental means. A heating system composed of allocated heating elements provides for a relatively homogenous temperature distribution, while the admissible temperatures determined by the hydraulic system are not exceeded. In order to approve the effectivity of the drafted heating system, tests under harsh outdoor conditions have to be performed.

References

[1] Gramsch, C., Blaszczyk, A., Löbl, H. & Großmann, S.: Thermal Network Method in the design of power equipment, Scientific computing in electrical engineering SCEE, Sinaia pp. 213-219, 2006.

[2] Böhme, H.: Mittelspannungstechnik, 2. Auflage, Huss-Medien GmbH, Berlin, 2005.

[3] Michejew, M.A.: Grundlagen der Wärmeübertragung. Übersetzung aus dem Russischen / von Edgar Scheitz Deutsche Bearb. / von Hans Liebsch. 3. berab. Aufl., VEB Verlag Technik, Berlin, 1961.

[4] Elsner, N., Fischer, S., Huhn, J.: Grundlagen der Technischen Thermodynamik. Band 2. 8. grundlegend überarb. und erg. Aufl., Akademie Verlag, Berlin, 1993.

[5] Baehr, H.-D., Stephan, K.: Wärme- und Stoffübertragung, 2. Auflage, Springer-Verlag, Berlin, 1996.

[6] Dahl, W.: Werkstoffkunde Stahl, 4. Auflage, Springer-Verlag, Berlin, 1985.

RAILWAY M201, SECTION KRIŽEVCI – KOPRIVNICA – STATE BORDER: UPGRADE AND CONSTRUCTION OF SECOND TRACK

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Abstract

Currently, there is an emphasis on upgrade of railway network in Croatia. This is especially visible on railways that are a part of pan-european corridors (Vb: Rijeka-Zagreb-Budapest and X: Salzburg-Ljubljana-Zagreb-Beograd-Niš-Skopje-Veles-Thessaloniki). Design and construction of these railway corridors is in large part funded by the European Union through IPA programme, Priority axis 1 of IPA Component IIIa. Upgrade of railway M201 (State border – Botovo – Koprivnica – Dugo Selo) is divided into 2 phases. Phase 1 (Dugo Selo-Križevci) is currently in final stages of design, and construction should start in 2014. Phase 2 (Križevci-Koprivnica-state border), which is the subject of this paper is in the first stages of design, with obtaining of construction permit due in 2016, which will be followed by construction that is supposed to be finished by year 2020. Upgrade of M201 on this 43.2 km long section mostly consists of construction of second track parallel to the existing one, with modernisation of the control command and signalling subsystem, and reconstruction and upgrade of existing stations, stops, road crossings and structures. Smaller parts of the existing railway track will be reconstructed, or even abandoned to achieve design speed and station layout requirements, all in accordance with the proposed traffic technology. This paper presents the technical solutions of railway track alignment, road crossings and structures, that are a basis for all other design work on this project. Also, current state of spatial planning documentation regarding this project and influence of other big infrastructure projects, especially motorway projects that intersect with the rail corridor will be shown.

Keywords: railway design, upgrade, EU funding, construction

1 Introduction

The project of the upgrade and construction of second track on railway line section Križevci – Koprivnica – National Border is defined in the Transport Operational Programme 2007-2013, which forms the strategic basis for the absorption of funds from component III of IPA's programme in the transport sector. The project encompasses the reconstruction of the existing track on the Križevci – Koprivnica – National Border section for nominal speed of 160 km/h and the construction of the second track alongside the reconstructed track of the railway line M201 (State Border – Botovo – Dugo Selo). The Design Contract was signed on Dec 10th 2012 by HŽ Infrastruktura d.o.o. as the Contracting Authority and URS Polska Sp. z o.o., in consortium with URS Infrastructure and Environment UK Limited and IDOM Ingenieria y Consultoria S.A., as the Consultant. The total value of the design contract is EUR 5.3 million, of which 85% is provided by the European Union and 15% by the Croatian Government. The deadline for the completion of the design is 42 months from the contract signature date.

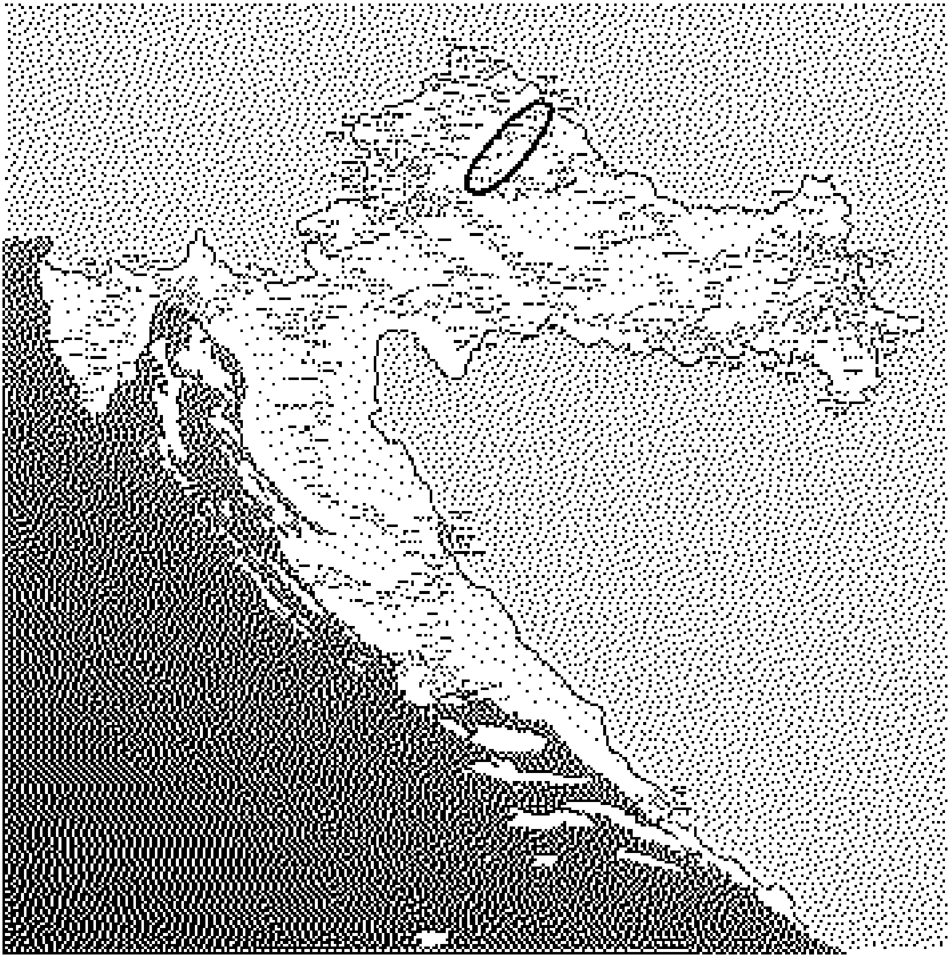

Figure 1 Position of Križevci-Koprivnica-SB section in Croatian rail network

The overall railway network corridor RH2 (State border Hungary/Croatia – Koprivnica – Dugo Selo – Zagreb Main Station – Karlovac – Rijeka – State border Croatia/Slovenia), as well as the railway transport corridor R**3 within the Republic of Croatia shall become a constituent part of the future Trans-European conventional railway transport network. Considering the geographical conditions, the characteristics of particular parts, the traffic-technological and organizational conditions and the planned technical conditions, the future railway transport corridor R**3 is divided into 4 sectors:

- Sector I ⇢ State border Hungary/Croatia – Koprivnica – Križevci – Dugo Selo
- Sector II ⇢ Zagreb railway node
- Sector III ⇢ Horvati – Karlovac – Skradnik – Drežnica – Krasica
- Sector IV ⇢ Rijeka railway node and Matulji – Borut – Pula/Raša/Slovenia

In all parts of the railway corridor RH2 it is necessary to conduct extensive works with an aim to increase the transport capacity, to reduce travel time and to align the condition and characteristics of railway infrastructure with the conditions of existing European railways regulation (Railway Safety Directive, Directive on the interoperability of the trans-European conventional rail system, Technical Specifications on Interoperability of trans-European rail system).

2 Existing state

The existing railway line M201 State border – Botovo – Dugo Selo on section Križevci – Koprivnica – State border partially passes through urbanized areas, and partially through uninhabited area. Although it partially passes through mountain area, according to its characteristics is a predominantly lowland railway section, at which the longitudinal gradient is less than 8 mm/m. At subsection Vojakovački Kloštar – Lepavina there are horizontal curves of 460 up to 715 m in radius which limit the designed speed at this subsection to 90 to 120 km/h, while on the remaining part of the line the design speed is 140 up to 160 km/h. The maximum allowed mass of trains on the entire railway section is 22,5 tons/axle and 8 tons/m' (corresponding to UIC line categorisation freight load model D4).

Figure 2 Existing railway near Mučna Reka

The usage condition is deteriorating year by year, even with recent renewal, thus the maximum allowed speed at subsection Križevci – Mučna Reka is reduced to 60 km/h, and at subsection Mučna Reka – Koprivnica – Botovo – State border at 80 km/h. The subsection Križevci – Koprivnica has been renewed in 2012, and subsection Koprivnica – Botovo – State border in 1978. The entire section has been electrified with AC 25kV/50Hz system, and is ensured with AB (automatic block), while relay devices are installed in stations. The traffic is operated in block intervals. The stations for planning traffic are Križevci and Koprivnica, and the maximum allowed train length, considering the usable main track length is 515/521 m. Limiting part of the railway line is the Križevci – Lepavina subsection, with average daily traffic of 46 trains in 2012. (31 passenger and 15 freight trains). Track capacity for Križevci – Lepavina subsection is 69 trains total, according to 2012./13. Croatian railways time table.

Table 1 Technical elements of the existing railway [1]

Minimum horizontal radius	452 m
Maximum longitudinal gradient (open track)	8.0 ‰
Maximum longitudinal gradient (stations and stops)	4.8 ‰
Minimum vertical radius	8000 m
Rails	49E1
Track formation width	6.5 m
Transversal gradient	5.0 ‰

The railway line is situated with its larger part on the embankment and with a smaller part in the cutting. Embankments are mostly low, 2-3 m high, except on the mountainous part of the track from Vojakovački Kloštar to Lepavina where the embankments are up to 15 m high. Railway line passes partly through agricultural area and settlements, and partly through forests. The embankments are sporadically narrow, low, and in some parts of inadequate width which causes dispersal of the ballast. At particular places, the bad condition of the substructure of the track can be seen. In spring of 2013, heavy rain caused landslides that caused closure of the railway line for extended periods of time. There are 6 stations and 4 stops on the Križevci – Koprivnica – state border section. Out of the 8 existing bridges, only Steel bridge “Drava-Botovo” is significant, with length of 291 m. The rest of the bridges are single span structures with length up to 24 m. There are multiple crossings with the existing road network, with 13 level crossings and 5 de-levelled ones (4 underpasses and 1 overpass).

3 Planned state

Design and construction of the Križevci – Koprivnica – State border section is separated into 4 subsections:
· II.a – Križevci (excluded) – Carevdar (included)
· II.b – Carevdar (excluded) – Lepavina (included)
· II.c – Lepavina (excluded) – Koprivnica (included)
· II.d – Koprivnica (excluded) – State Border

3.1 Railway

Given the requirement that the future double track railway should be constructed for nominal speed of 160 km/h, changes to the existing horizontal track geometry were necessary. Major changes to track geometry were done on a 2.5 km long section directly after Križevci station exit, and on a 4.5 km long section between Carevdar and Lepavina. On the rest of the track, only existing horizontal curves R ‹ 1700 m needed to be reconstructed.

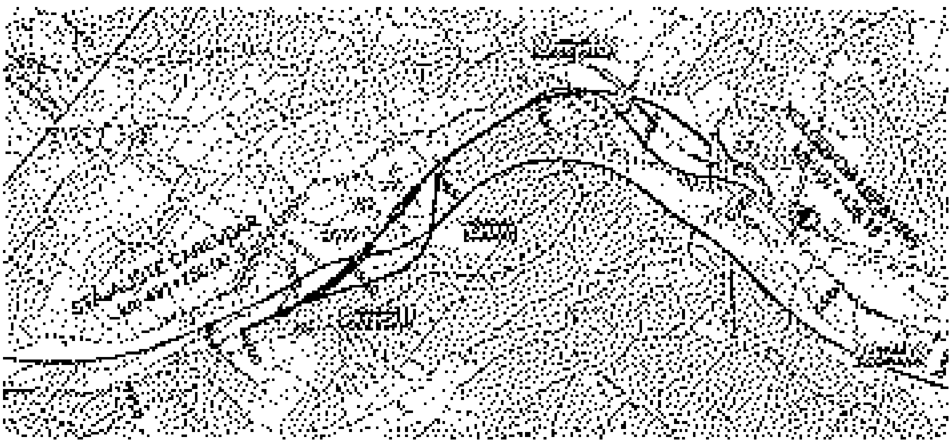

Figure 3 New track geometry between Carevdar and Lepavina

Maximum longitudinal gradient doesn’t exceed 10 ‰ on open track sections and 2.2 ‰ in stations. On the whole length, the railway line is designed as a continuous welded track with 60E1 rails, with 4.5 m distance between tracks on open sections and 4.75 m in stations. The rails are installed on prestressed concrete sleepers, 60 cm apart, which are set on crushed stone ballast layer with minimum thickness of 40 cm. Rail and sleepers are connected using elastic type fastening.
Existing Lepavina and Koprivnica stations will be reconstructed, Mučna Reka station will be downgraded to stop, and Drnje station will be abandoned after the construction of a new station Novo Drnje which is planned app. 1 km in Botovo direction. In addition to existing stops, a completely new stop will be constructed near Peteranec. After the reconstruction, maximum allowed train length in stations will be 750 m for freight and 400 m for passenger trains.

Table 2 Planned stations and stops after the reconstruction [2]

No.	Name	Status	Km position	Number of tracks
1	Majurec	stop	485+454.85	2
2	Vojakovački Kloštar	stop	480+961.07	2
3	Carevdar	stop	491+750.00	2
4	Lepavina	station	497+436.16	6
5	Sokolovac	stop	499+580.64	2
6	Mučna Reka	stop	504+123.63	2
7	Koprivnica	station	510+668.89	14
8	Peteranec	stop	515+948.27	2
9	Novo Drnje	station	520+197.89	6
10	Botovo	station	522+762.51	3

3.2 Structures

After inspection of existing railway structures on Križevci – Koprivnica – State border section, it was concluded that unfortunately most of them can't satisfy current norms and regulations. Because of this only existing road underpass in Koprivnica, and pedestrian underpasses in Koprivnica station will be renewed and maintained. All other structures will be either demolished or abandoned, in regard to changes in horizontal track alignment. On Carevdar – Lepavina section, construction of 2 new viaducts, 350 and 635 m long is planned. They will also be used as animal passages. Also, 2 structures are planned as animal crossings, one under and the other over the railway line.
Parallel to the existing one, a new double track bridge over Drava river will be constructed. The new steel bridge is designed with a main span of 145.5 m, and 2 smaller spans of 72.25 m, on on each side of the main span.

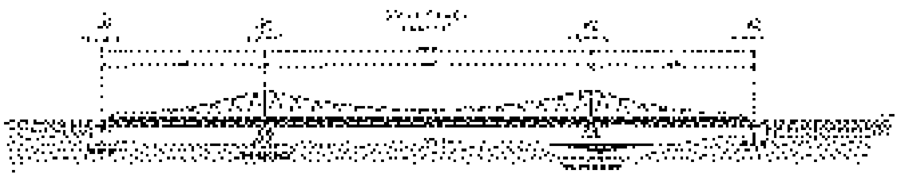

Figure 4 New Drava bridge

Given the demands of the new legislature that states all existing rail crossings with state or county roads must be de-levelled in case of rail reconstruction, 8 new road overpasses and one road underpass will be constructed.

3.3 Road crossings

Given the changes in horizontal track geometry at exit of Križevci station and between Carevdar and Lepavina, and planned construction of DC10 and Podravska motorways, there are 30 crossings with existing and planned roads (not calculating in field paths).

3.3.1 Existing roads

The reconstructed railway crosses a total of 26 existing roads. After construction, there will be 9 overpasses, 3 underpasses and 2 passages under bridges or viaducts. 6 of the existing level crossings with local or unclassified roads will be kept in level, with reconstruction of access roads and security systems (signals, half-barriers). 6 of the crossings at the sections where the track is set in a new corridor will be reconnected to the nearest de-levelled crossing. In front of Lepavina station, new railway corridor crosses the existing DC41 state road at 2 positions, so two road overpasses (Lepavina 1 & 2) will be constructed. The fact that railway and state road corridor are parallel on long stretches of the section, made technical solutions of de-levelled crossings much more complex, especially for crossing of county road 2212 near Križevci and Pavelinska ulica in Koprivnica.

3.3.2 Planned roads

There is a total of four crossings with planned motorways, three of which are with future state road DC10, former highway A12. The crossing near Majurec is a part of Križevci-Kloštar Vojakovački section of the motorway. This section has a construction permit, and the crossing is solved with a road viaduct. Rail reconstruction on this part was designed to use the predicted free space in one of the spans. Two of the crossings with DC10 are on Kloštar Vojakovački-Mučna Reka section of the motorway. Since there are no permits issued for this section, railway design was done in coordination with Croatian Motorways and Croatian Roads to keep the necessary future changes of motorway design to minimum. Near Peteranec, planned Podravska freeway will cross the railway corridor on a road viaduct. Since that road is still in the conceptual design stage, and only construction of second track is planned on that part of the section, there were no major issues with this collision.

4 Spatial plans

Base for the design of this railway section was conceptual design made in 1998. Unfortunately, the rail corridor from that design was never implemented in the spatial plans. Because of this, changes made to the DC10 motorway corridor in 2011 collided with the M201 conceptual design. Also, new railway corridor proposed by in the conceptual design had big influence on the existing Orthodox monastery near Lepavina.

Because of these issues, a new railway corridor between Carevdar and Lepavina was designed, taking into account proposed corridors in Sokolovac municipality spatial plan. This corridor, as well as the new railway corridor at the beginning of the section which was a part of Križevci spatial plan, were never implemented in the county spatial plan.

This meant that before the approval of the Environmental Impact Assesment and subsequent request for issuing of Location permit, County spatial plan needed to be changed. This process was started in June of 2013, and should be ended by May of 2014.

In addition to this, reconstruction of existing road crossings demanded the targeted changes of lower level spatial plans in City of Križevci and municipality of Sokolovac. Procedure was started in December of 2013, and should be finished in time for Location permit extraction.

References

[1] HŽ Infrastruktura: Terms of Reference, Zagreb, 2012.

[2] URS Polska Sp. z o.o., URS Infrastructure and Environment UK Limited and IDOM Ingenieria y Consultoria S.A.: Željeznička pruga M201 za mješovit promet državna granica - Botovo - Dugo Selo (poboljšanje postojećeg i izgradnja drugog kolosijeka), Preliminary design, Zagreb, 2014.

[3] Dvokut ECRO d.o.o.: Željeznička pruga M201 za mješovit promet državna granica - Botovo - Dugo Selo (poboljšanje postojećeg i izgradnja drugog kolosijeka), Environmental Impact Study, Zagreb, 2014.

[4] Spatial plan of Koprivničko-Križevačka county, SG8/01,8/07,13/12, 2001.-2012.

[5] www.hzinfra.hr

[6] http://www.uic.org/spip.php?article2897

TRAFFIC-CONSTRUCTIONAL ASPECTS FOR BUILDING OF BYPASS AROUND NIS IN CORRIDOR X

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Abstract

City of Niš is the crossroads of the most important Balkan and European traffic routes. The territory of the City is intersected by mportant directions: Corridor X – road connect the Balkan with Central and Western Europe, Main lines E70 and E85, and regional and local traffic network of roads and railways. Niš urban and demographic development has not been matched by investment in road and railway infrastructure, consequently this network are overloaded. Unadjusted development of the city on already built of railway lines, led to the spatial and functional conflicts and many negative effects. Building of bypass line includes the implementation of Corridor X through node and unobstructed transit traffic on Corridor X. This creates the conditions for the separation of passenger and freight transport and relieving of freight traffic the central zone of the city of Nis. The construction of the railway bypass around the City of Niš will allow implementation of EU standards for interoperability, will contribute to the further development of the city and will also contribute achieving better conditions for transport of passengers and freights thus helping divert international road traffic to the railways with the consequent benefits in environmental terms. Aim of the author work is to show traffic and civil aspects of the construction of the bypass, its interdependence and the impact on traffic Niš junction. Optimal technical and spatial – urban solution can be realized in phases to meet the needs of transport and development plans of City of Niš.

Keywords: railway bypass, traffic, infrastructure

1 Introduction

Disharmony of city development, traffic and transportation infrastructure leads to spatial and functional conflict in the city area of Niš. Problem of railway traffic for the city can be at one side – a limiting factor, and at the other – a stimulating factor of the development. Functionality of railway transportation in the city should be subsumed under the needs and requests of the citizens, but in the functioning of the railway transportation system in the Serbia. Reduction of the collision points, city–railway should correspond to the previously planned and defined strategy of the city development. It includes:

· priority of phase construction and modernization of the railway in the Niš;
· dynamic plan of activities;
· control of activities.

The problem of level crossings in the center of Niš is especially expressed and endangering the safety of users and complicate realization of all transportation types. In the present state, passing freight trains through the central zone of the city produces increased levels of pollution and noise, with an increased risk of environmental protection because of the passage of hazardous materials through the city. The general aims of railway bypass construction around the city of Nis are:

- harmonization of transportation infrastructure development with the development of the city;
- harmonization of transportation infrastructure development with the development of the airport of Niš;
- relocation of freight transportation from the city center;
- the development of railway systems without disrupting the transit transportation on Corridor X with local work.

Settings to define the technical and technological parameters of railway bypass construction are:

- official planning documentations;
- spatial limitations and places of collision with existing and planned installations;
- spatial limitations and places of collision with existing and planned transportation;
- necessary capacities and facilities at the stations on the railway bypass;
- connection of new railway bypass with existing infrastructure capacities of the node;
- connection of new railway bypass with existing railway line Nis – Dimitrovgrad;

2 The current state of the transportation system of the Niš.

2.1 General features – location and system elements

Niš as a regional, administrative, cultural, educational, industrial and transportation centre at the international network is a place of connection and separation of the two main international directions E-85: Budapest–Athens and E-70: Paris–Sofia. In the transportation system of Niš there are all forms of transportation, for the various types of passenger and freight transport, what Niš makes an important connection in the link of passengers and goods flows. From the aspect of the road traffic, Niš is the crossroad of international regional and local road traffic. Backbone of bypass roads around Niš are sections of the international road E-75 (M-1) and E-80 (M-1.12). In progress is construction of the highway E-80th. From Niš are separating 11 routes on which are organized bus transportation.
From the aspect of air traffic, city airport "Konstantin Veliki" is qualified for receiving medium aircraft. "Konstantin Veliki" is the second category airport, and it is airport for regular and charter flights for the passengers of Niš and the region, and serves as an alternative airport for Belgrade, Skopje and Sofia. Using of the runway is limited by the existing electrified railway.

2.2 Railway transportation diagrams

Niš railway junction is one of the most important and oldest junctions, created more than 100 years ago and has a very important role in the Serbian railway infrastructure, also has great influence at the development of the city Niš. Railway infrastructure capacities in the junction Niš were built and modernized in stages and gradually, and parts are:

- main lines: Beograd–Niš–Preševo–border, Niš–Dimitrovgrad–border;
- railway line of the first order (Niš)–Crveni krst– Zaječar–Prahovo Port;
- inside the junction connections are made over railway lines: Crveni Krst–Niš Marshalling Yard, (Crveni Krst)–(Ćele Kula), Trupale–Niš Marshalling–Međurovo and Niš–Junction Bridge–(Niš Marshalling);
- railway facilities in the junction: triangle Crveni Krst–Ćele Kula.

Circular connection has multiple role because of the connecting station Niš Marshalling Yard with the depot and industrial tracks in the station Crveni Krst, station Niš marshalling with station Niš and Mechanical industry and station Niš with depot in the station Crveni Krst.

All mentioned tracks are electrified, category D-4 and load 22.5 t/ax and 8.0 t/m', except track Niš–Dimitrovgrad–state border (D-3, 22.5t/ax and 7.2 t/m') and track (Niš)–Crveni Krst–Zaječar–Prahovo Port (B-2, 18.0 t/ax and 6.4 t/m').
All railway sections are equipped with APB, except sections Niš–Crveni Krst and Crveni Krst–Matejevac. For interstation distance Niš–Crveni Krst, railway is double track and it is equipped with APB devices for two-way traffic. According to schedule for 2012/13 maximum allowed speeds are between 30 km/h and 70 km/h.
Existing junction Niš (Figure 1) consist stations: Niš, Niš Marshaling Yard, Crveni Krst, Trupale, Međurovo, Matejevac, Ćele Kula and Niška Banja. Other halts are: Palilulska Rampa, Vojna bolnica, El Niš, Pantelej and Rasputnica Most. The main passenger station is the station Niš, the main marshalling yard station is Niš Marshaling yard.

Figure 1 Niš junction

2.2.1 The organization of passenger transportation

All stations are opened for the reception and departure of passengers. At the station Niš all categories of trains for passenger transport have stopping. Besides the transit trains all trains for passengers are starting/ending ride in the station Niš. Transportation flows are presented according to the schedule for 2012/13th year. Marching of trains for passenger transport is shown in Figure 2. At the station Niš are performing necessary technological operations (changing of locomotives, adding or removing a wagons, combining of train set and etc.), and for relation Belgrade–Dimitrovgrad at the station Niš are changing the traction type and driving direction.

2.2.2 The organization of freight transportation

The current intensity of freight transportation is relatively small, partly because of the lack of adequate series of wagons and locomotives. All trains in the domestic traffic are starting or ending driving in the Niš Marshaling Yard station. Traffic flows are presented according to the schedule for 2012/13th year. Marching of freight trains is shown in Figure 3.

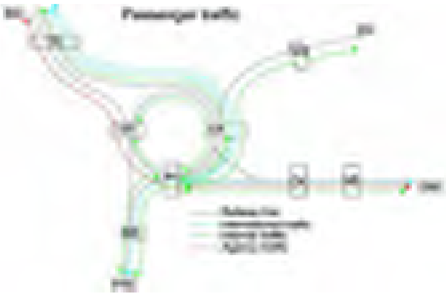

Figure 2 Passenger traffic

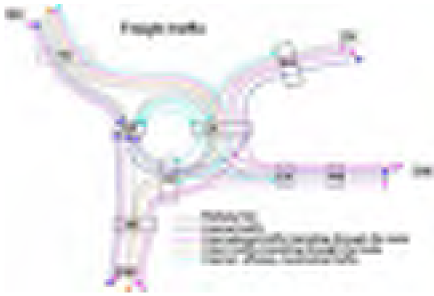

Figure 3 Freight traffic

At the station Crveni Krst are performing change of locomotives and other necessary technological operations for trains for/from the direction of Dimitrovgrad. Local work with vehicle goods is not concentrated in one place and its performing at the station and industrial tracks. Transport of part-load cargo and luggage is minimal.

2.2.3 Organization of shunting works

Shunting work on the composition and decomposition of freight trains is performed at the station Niš Marshalling Yard and work on forming of passenger wagons at the station Niš. In the junction Niš is performing shunting work in order to service loading and unloading tracks, industrial tracks, serving of stations, etc. Shunting works performs more difficult because of the shunting locomotives lack.

2.3 The railway system infrastructure

Niš is the second largest city in Serbia, with all the functions of the regional center and its geographical position is located at the intersection of traditional transport corridors of Western and Central Europe to Greece and Asia. Direction that made basis of today's railroad junction are north–south (Belgrade–Skopje) and west–east (Niš–Sofia). The urban area of Niš is topographical located at river Nišava's valley and spatially has developed along the Niš valley and hence the first built big infrastructure objects: railway line Niš–Sofia and international road Niš–Sofia. Accelerated process of industrialization and urbanization, there, has been spatial expansion of settlements that has taken the functions of urban character, so that the railway infrastructure at this moment is located in the very urban area of Niš. Main stations are:

- Niš;
- Niš Ranžirna / Niš Marshalling Yard;
- Crveni Krst.

2.3.1 Station Niš

The station Niš is the main passenger station, with five platforms for reception–departure tracks and one bypass track. Platform next to the station building and two island platforms are connected with underpass and have awnings. The station Niš also performs technical tasks of passenger station. The station serves for the passage of freight trains on the direction Niš–Dimitrovgrad.

2.3.2 Station Niš Marshalling Yard

The station is marshalling yard and sorting on the network YR, in the parks with parallel tracks, turnout track and small marshalling yard hill.
Station tracks are classified into the group:
· Reception–departure group has 8 tracks;
· Marshalling yard–departure group has 18 tracks;
· Station track group has 8 tracks;
· Bypass tracks (3 tracks).

The total length of the station tracks is 25920 meters, with 90 single and 8 English switches. Within the station Niš Marshalling Yard is located depot and wagon workshop. From the station stands out industrial track "VP 3472." It was designed for a former military airport in the Niš.

2.3.3 Station Crveni Krst

The station Crveni Krst is interstation on the railway Belgrade–Skopje and it is connecting station for railway Niš–Zaječar. Indirectly, through triangle, is connected with the railway Niš–Dimitrovgrad. Over circular railway, Crveni Krst is connected with the stations Niš Marshalling Yard and Niš.
On both sides of the station Crveni Krst is connected depot and from the depot is connection with the Factory of switches, which is located on the right side of the tracks. On the left side of the tracks from station stands out tracks for industrial zone of Niš. From the railway station Crveni Krst stands out 12 industrial tracks that serve for loading and unloading of goods, within of owners and users of industrial tracks.
For tasks that are performed by the station Crveni Krst there are 10 tracks. The total length of the station tracks is 6415m, with 29 single and 7 English switches. Depot group of tracks have about 6800m and 30 switches.

3 Spatial limitations for development of railway infrastructure in the Niš

3.1 Planning regulations

RS Spatial Plan 2010–2020 as a document of the highest importance envisages development of railway infrastructure in the Republic of Serbia, and special significance is given to the planning, construction, reconstruction and modernization of railway lines, as well as stations and other facilities, for performing passenger and freight transportation on Corridor X.
Spatial plans of the Republic determines the concept of development, organization, protection and usage of the area covered by the plan. It is implemented by developing of planning solutions and plans for special purpose of areas, spatial plans of local governments, urban planning and other plans. In all relevant plans for the city of Niš, as well as the general development plans it is recommended the relocation of railway freight transportation from the city center.

3.2 The functionality of transportation

Optimal technical–technological and spatial–urban planning solutions for railway bypass can be realized in phases according to the requirements of transportation and development plans of the Niš.

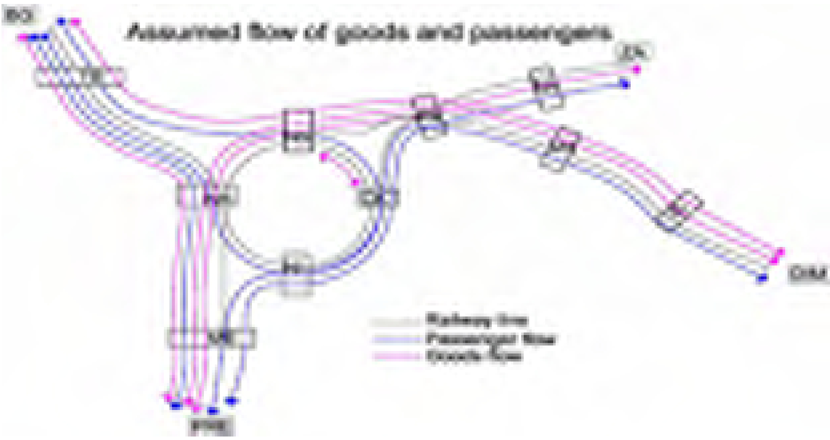

Figure 4 Assumed flow of goods and passengers

Construction of the bypass is the first step of undisturbed passage of double-track railway on Corridor X and undisturbed passage of transit traffic on Corridor X through the area of the Niš. This creates the conditions for the separation of passenger and freight transportation and relieving of the city central zone of freight transportation.
Organizational and functional solution of railway transportation is conceived with position of the stations Niš and Niš Marshalling Yard within of Niš railway junction. Technological requirements on which are based variants of solving railway bypass around Niš are:

- using of the existing railway corridors and land to the most possible extent;
- keeping the railway capacities on existing locations with modernization and integration into a new solution, all in accordance with the spatial and urban plans;
- the separation of the freight and passenger transportation and the releasing of the city central zone of the freight transportation;
- direct trains traffic for passenger transportation on main lines through the station Niš without changing the direction of the ride and head of the train, as well as changes of the traction type;
- the high level of efficiency and safety of railway transportation;
- functional and rational solution for railway bypass and stations as well as the connection of a new bypass to the existing railway infrastructure facilities;
- providing the necessary capacities, passing capacity and transportation safety, fast passage of trains, passengers and goods through the junction, the maximum protection of the environment and to minimize investment and exploitation costs, and all in accordance of the city development and the economy;
- separate grade intersection of the railway with road traffic;
- retaining and improving railway connections with existing and planned industry;
- compatibility of railway, road and air transport in the city of Niš.

3.3 Construction feasibility

In accordance with technological requirements, spatial and planning restrictions is narrowed the choice of alternative solutions of the route of a new railway bypass. Corridor of the route with minimum variation is placed on the north of the city periphery. Beginning of the railway bypass is on the output switch at the station Niš Marshalling Yard over existing halt Pantelej and along the highway corridor, on the left side and connection to the existing alignment of the railway Niš–Dimitrovgrad, to the station Sićevo.

The basic elements of the route should be take into account the following:
- reconstruction and modernization of the railway double track bypass should follow the basic settings according to the European Agreement on Main International Railway Lines (AGC), European Agreement on main lines for combined transport (AGTC), and the composition of the Trans–European railway network in mixed traffic and high speeds;
- technical standards for interoperability ECTS;
- railway transportation will be developed to a high level of service for the passenger, freight and combined transport.

The goal is to define the long-term of optimal technological and spatial urban solutions that can be implemented in stages and phases, in accordance with the geostrategic importance of the national and European railway network and transportation requirements according to European standards, and in accordance with the requirements and development plans of the Niš as an important regional city by European standards.

4 Construction of the railway bypass around Niš and the effects of construction

Bypass railway starts from the output switch of the station Niš Marshalling Yard. Part of the railway track from station Niš Marshalling Yard to the station Crveni Krst will be relocated and it will be built separate grade intersection (above or below) with the existing railway Trupale–Crveni Krst. Bypass route continues on the corridor defined by the spatial plan, the northern periphery of the city, to the connection of the existing railway line Nis–Dimitrovgrad in the area of Prosek. Length of the new railway bypass is about 19 km. On railway bypass is planned to build new stations: Niš North, Pantelej and Vrežina.
Between stations Trupale and Crveni Krst, at bypass line, is planned to build the station Niš North. The capacity of this station will be designed to satisfy the requirements of transport and transportation, with the corresponding groups of tracks designed to receive and departure of trains. From this station it is possible to make connection with the industrial tracks that are in the station Crveni Krst. This location, because of the roads and airport proximity has advantages for the eventual construction planning of goods–transport center. Neighboring stations are Trupale, Niš Marshalling Yard, Crveni Krst and Pantelej.
The new station Pantelej, at bypass, in the transportation meaning, it will be junction railway station to Zaječar and it is designed for receive and departure of trains. The effect of its construction is reflected in the separation of passenger and freight transportation. Neighboring stations are Niš, Crveni Krst and Vrežina.
The new station Vrežina is designed for receiving and departing of trains in all categories. The effect of its construction is to increase passing capacity at new bypass line. Neighboring stations are Prosek and Sićevo.

5 Conclusion

According to the adopted goals of reconstruction and modernization of railway lines in the city area, taking into account of national and international importance of transportation infrastructure in the Niš, construction of the railway bypass around Niš is the first phase of modernization. The construction of railway bypass can be performed out in stages. Phasing of construction should not affect the continuity in the execution of the traffic flow. New stations on the route of the railway bypass which will be formed in the first phase with minimal infrastructure capacity in the next phases of development would allow railway transportation to competitive advantage over the different types of transportation in the terms of safety, shorter travel times, massive transportation, comfort, cost and cost-effectiveness.

References

[1] Spatial Plan of the Republic of Serbia 2010-2020.

[2] The Spatial Plan for the Niš – Bulgarian Border Infrastructure Corridor

[3] The Regional Spatial Plan for the area of Niš, Toplički and Pirot District

[4] The Spatial planning and Urban Planning documents of Niš

[5] The Spatial Plan for the administrative area of the city Niš

[6] Master Plan of Niš 2010-2025

[7] The General Regulation Plan for the city's municipal area Crveni Krst – I phase

[8] The General Regulation Plan for the city's municipal area Pantelej – I phase

[9] The General Regulation Plan for the city's municipal area Pantelej – II phase

[10] Detailed Regulation Plan of Nis – Dimitrovgrad, section Crveni Krst – Prosek, the highway corridorE-80 in Nis

[11] Available existing project documentation for Railway lines around Niš, Niš node, Niš aeroport and highway E-80 and E-75

REHABILITATION OF RAILWAY LINES ŠAMAC – SARAJEVO AND SARAJEVO – ČAPLJINA

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Abstract

When warring parties signed the "General Framework Agreement for Peace in Bosnia and Herzegovina" back in 1995, numerous railway bridges were destroyed, railway tracks mined, station buildings demolished and most of the rolling stock and locomotives destroyed or severely damaged. "The Study on the Transport Master Plan in Bosnia and Herzegovina" (JICA for World Bank, 2000) estimated (aggregated) value of direct damages to the railway sector in the country at 1,000 million US$. Within the "Emergency Transport Reconstruction Program", major railway capacities were reconstructed / rehabilitated, enabling the re-establishment of railway operations in Bosnia and Herzegovina. In the next stage, International Financial Institutions strongly supported not only institutional capacity building but also the drafting of relevant Studies with the primary objective of defining priority projects for the railway sector. Two railway lines connecting Bosnia and Herzegovina's capital, Sarajevo with the Croatian port of Ploče in the south and the Hungarian Capital Budapest in the north are included in the "5th Pan-European Transport Corridor" (branch c) and "SEETO Comprehensive Network" as well. This paper focuses on railway lines in the 5th Pan-European Transport Corridor (Šamac-Sarajevo and Sarajevo-Čapljina) rehabilitation projects from 2005 onwards. Being active participants in some of the projects, the authors had the opportunity to be involved in discussions with representatives of the Beneficiaries and the Banks, to conduct field surveys and comprehensive desk research as well and some of their findings are represented herein.

Keywords: railways rehabilitation projects, Corridor Vc, analysis

1 Introduction

Bosnia and Herzegovina (hereinafter: BiH) is a middle-income country, with small-size economy where approximately two thirds of the GDP has been created in the service sector, 25 percent in industry, and less than 10 percent in agriculture, hunting, fishing and forestry. It is located in South-East Europe or more precisely in South-West Balkans (see Figure 1) where the Dinaric Alps "cover" most of the land. Therefore, river valleys and mountain passes have been the most used transport routes since the time of old Romans.
Such a specific configuration of landforms and a relatively good geostrategic position have been the most influential factors in the development of transport systems in BiH. Moreover, the historical development of railways in particular was strongly influenced by the political and economic environment for mining and extraction of different ores (iron, coal, minerals) and other natural resources (e.g. wood, salt etc.) of BiH.
The railways have played different roles in the country's development during different historic periods. Still, the two most significant periods occurred during the Austro-Hungarian reign (expansion of the narrow gauge railway network) and when BIH was one of the six Social Republics within the Socialist Federative Republic of Yugoslavia. Anyway, the first subheading of

this Introduction provides more basic facts on the historical development of railways in BiH. The railways today remain a significant economic asset of BIH's economy and railway transport is still essential for operation of big industrial systems (e.g. power plants, steel factory, aluminium factory etc.). In addition, the railway system is connected to the Port of Ploče in Croatia in the south and the European railway network in the north, providing perspective for all export oriented companies in the country. The second subheading gives a brief review of BiH railways institutional setup.

Figure 1 Map of Bosnia and Herzegovina

EU accession was declared as a strategic priority for BiH hence the two sides signed the Stabilization and Association Agreement (SAA) – Interim Agreement, on 16th June 2008. Unfortunately, for BIH politicians' acceptance of this priority was (to use "legal terms") clearly just declaratory as SAA has never been affirmed. That is why this Introduction ends with a summary of common activities of the two sides (BiH and EU), having more or less the railways in its focus.

1.1 A Brief History of Railways in Bosnia and Herzegovina

The first railway line in BiH was constructed in the XIX century, during the Ottoman Empire reign. Construction started in 1869 and was completed in December 1872, when a 101.6 km long standard gauge railway line between Dobrljin and Banja Luka was put in operation. It should have been a section of (the never completed) 2 500 km long railway line Istanbul – Wien.
From 1878 to 1918, BiH was a part of the Austro-Hungarian Empire. During the first two years of this period, railway line Dobrljin-Banja Luka was rehabilitated and in 1882 (via Sisak) it was linked with the rest of the Austro-Hungarian railway network. Moreover, Austro-Hungarian Empire constructed 1611 km of narrow gauge lines (760 mm) all over the country (see Figure 2).
Between two World Wars, BiH was part of the Kingdom of Yugoslavia, and from 1918 to 1941 another 65 km of standard gauge lines (from Bihać to Bosanski Novi) were constructed as well as another 90 km of narrow gauge lines (from Trebinje to Bileća and from Ustiprača to Foča).
From 1945 to 1992 BiH was one of the six federal units in SFRY and the BiH railways network was managed by "ŽTP Sarajevo" (one of ex Yugoslavia's Railways Community units). During that period, more than 850 km of standard gauge railway lines (including the 87 km long

double track section Doboj-Zenica) were constructed, some of the narrow gauge lines were upgraded to standard gauge (e.g. Sarajevo-Doboj), but the best part of narrow gauge lines network was closed.
It was the time of contemporary railway technologies introduction (e.g. automatic block signalling, remote control of electric power consumption and so on) when 72% of the standard gauge railways network in BiH was electrified (25 kV AC, 50 Hz) as well. Consequently, in 1985, “ŽTP Sarajevo” recorded the best railway performance ever (19.1 million of passengers and 32.1 million tons of goods transported). Unfortunately, that was the end of a golden era for railways in BiH.

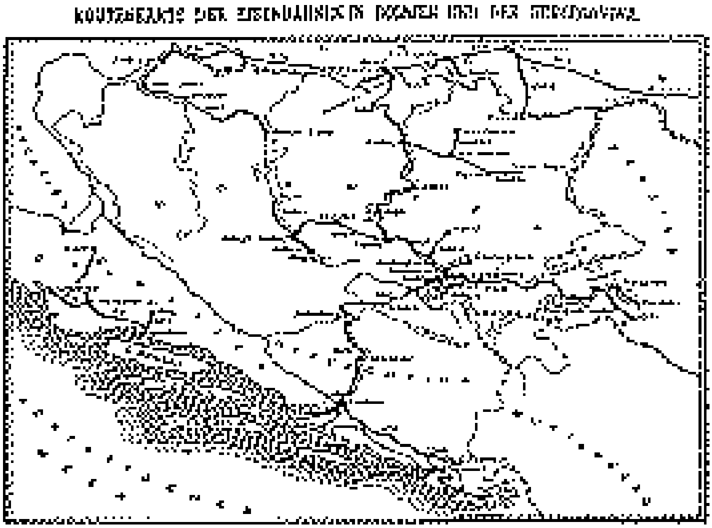

Figure 2 Map of Railway Network in BiH (1918)

In March 1992, BiH declared independence from SFRY. This act was followed by the beginning of wartime activities in the country, which lasted until November 1995, when warring parties signed the “General Framework Agreement for Peace in Bosnia and Herzegovina” (DPA). During the wartime, most of the important railway bridges were destroyed, numerous railway tracks were mined, buildings demolished, and major parts of rolling stocks were destroyed or severely damaged. Essential facilities and equipment (signalling, telecommunications, electric power supply etc.) were heavily damaged as well.
During the post-war period, within the “Emergency Transport Reconstruction Program”, (financial support was provided by International Financial Institutions, the European Union, and other bilateral donors), major railway capacities were partly rehabilitated, just enough to re-establish railway operations in the country. In 2001, the Study on the Transport Master Plan in Bosnia and Herzegovina estimated the aggregated value of direct damages to the railway sector at 1,000 million US$.
Today, the BiH railway network is poorly developed as there are just over two km of railway line per 100 km^2 (See Figure 4). On the other hand, all industrial systems are linked to the railways and most of BiH’s population lives in areas which railway lines traverse. Nevertheless, the only sure thing is that there is a huge perspective for development of railways in the country. However, this paper shall put focus on the improvement of key railway infrastructure in the country. Since 2000 when the “Phare Multi-Country Transport Programme” gave overall priority to railway lines in branch c of the 5th Pan-European Transport Corridor (hereinafter: Corridor 5-c), as numerous studies (e.g. TIRS, REBIS and so on) confirmed its importance not only for BiH's economy but also for the regional transport system development.

1.2 Institutional Framework

According to the DPA, the Inter-Entity Boundary Line divides BiH into two Entities: Federation of Bosnia and Herzegovina (FBiH) and Republika Srpska (RS). In addition, in 1999, Brčko District of BiH was established as a single administrative unit of local self-government existing under the sovereignty of BiH (and owned by two Entities).

The complex organizational structure of the country resulted with even more complex institutional establishment of railways: The owners of railway infrastructure in BiH are both entity governments as well as the government of Brčko District (each on its territory). Therefore, the railways in BiH are the "subject of interest" for three different ministries: At state level, there is the Ministry of Communications and Transport (MoCT) and in addition both Entities have Ministries of Transport.

Furthermore, there are two railway undertakings established by the Entities, which are at the same time infrastructure managers and railway operators: Railways of Federation of BIH (ZFBH) and Railways of Republic of Srpska (ZRS). Both companies are members of the "Union Internationale de Chemins de Fer" (UIC). Consequently, operations are regulated in legal terms by the State and Entity Laws, but also by the specific railway regulations which elaborate in detail each component of the railway transport system.

Finally, pursuant to Annex 9 of the DPA, the Entities founded a joint public corporation as an inter-entity umbrella organization for all stakeholders. A Management Board (one general manager and two deputies) and a Board of Directors (representatives from both Entities as well as from both railway undertakings) were established in line with the Agreement on establishment of the Bosnia and Herzegovina Railways Public Corporation (hereinafter: BHRPC).

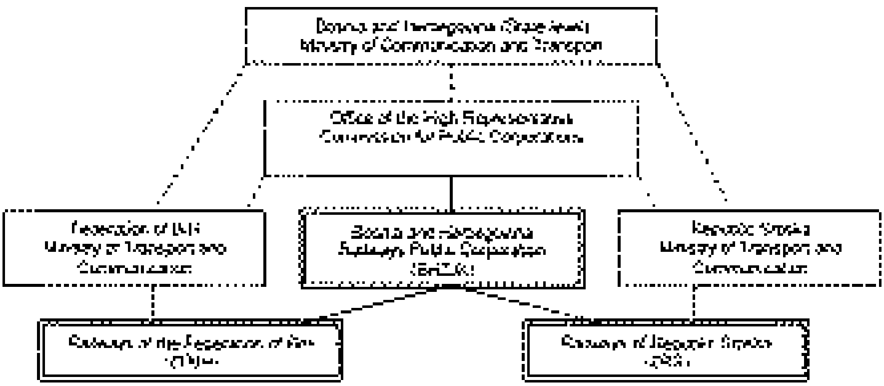

Figure 3 Basic structure of railway sector in BiH (DB International, 2008)

According to the Agreement on establishment, BHRPC mission is to ensure institutional cooperation (on inter-Entity level) and to act as an overall supervisory and regulatory organization by (i) allocating train paths for inter-Entity and international traffic (ii) setting railway infrastructure investment and rehabilitation priorities (iii) examine the ways and means to improve the level of service on the two main lines, (iv) harmonizing technical systems such as signalling, safety and telecommunication, (v) harmonizing of fee structures and levels, (vi) settling of accounts between the railway undertakings, and (vii) harmonizing regulations and standards to EU and international organizations requirements.

Furthermore, the Agreement confirmed the responsibility of the State for international and inter-entity transport and consolidates the role of the BHRPC as an agent for international financial assistance and for communication with international organizations and the EU. Finally, the Agreement stipulates that BHRPC can act as common agent for all matters which will contribute to the development of the railway sector in BiH. However, both the representative

and the active role (in international activities) are subject to the approval of the Entities' governments or the railway undertakings.
The state Railways Act stipulates the establishment of the Railways Regulatory Board (hereinafter: RRB) in BiH as part of BiH MoCT financed from the state budget. RRB activities are related to: (i) technical standards specification, (ii) railway sector monitoring and (iii) licensing of safety certificates, inspection and railway accidents statistics. RRB is still not fully operational, because a lot of functions have been delegated without adequate personnel and financial support.

1.3 BiH and EU in context of Railway infrastructure rehabilitation

The Declaration on Special Relations of BiH and EU was adopted in 1998. In 2006, the BiH Directorate for EU integration drafted "EU Integration Strategy", where "Parliamentary Assembly adopted Conclusions which, inter alia, present the presence of a full political consensus that EU membership is the highest possible priority for BiH."
This act was followed by drafting of the "Transport Sector Policy and Strategy for BiH" (supported by EBRD and WB) to the MoCT. Both documents were completed in 2007 and adopted by the Council of Ministers (hereinafter: CoM) back in 2008. Unfortunately, the process of its acceptance in the Parliament took more than five years and recently the documents were returned to the CoM for its additional adjustment.
An integral part of the signed SAA is the "Protocol on Land Transport", which should ensure that land transport between and through the territories of BiH and EU is developed in a coordinate manner. The Protocol includes a section titled "Rail and Combined Transport" stipulating the adoption of coordinated measures by BiH and EU for the development and promotion of rail and combined transport. In addition, this section of the Protocol also refers to the readiness of EU to support the infrastructure development in BiH through its financial institutions and lending instruments.
On June 11th, 2004 BIH was one of the countries to sign Memorandum of Understanding for the development of the Core Regional Transport Network (SEETO Comprehensive Network). This act established South East Europe Transport Observatory (SEETO) as regional transport organization. One of the main SEETO objectives is to enhance local capacity for the implementation of investment programmes.
In the meantime BHRPC has implemented "Regional project Railways in BiH l" (63.5 million € loans from EBRD & EIB and donations from Government of Canada and Government of Japan) and "Regional project Railways in BiH ll" (173 million € from EBRD & EIB loans, grant EU and from BiH budget). "Regional project Railways in BiH lII" announcement is expected soon. Moreover, EU continuously provides technical assistance to railway authorities in BiH (e.g. through IPA I Action Programmes).
One of the most important projects of railway sector institutional building in BiH, so far was the harmonisation of regulations on maintenance of railway infrastructure and rolling stock with the EU Directives (2001/16/EC on interoperability and 2004/49/EC on safety) and Technical Specifications for Interoperability (TSI). "Updated and harmonised railway regulations concerning the railway infrastructure shall provide the regulatory basis for BHRPC for implementation of design, upgrade and construction activities concerning the railway infrastructure in BiH."
On the other hand, Infrastructure Projects' Facility of The Western Balkans Investment Framework (WBIF) has already provided technical assistance for two railway infrastructure rehabilitation projects in BiH (TA2-BIH-TRA-02 and WB5-TRA-BIH-14). WBIF is a joint initiative of the EU, International Financial Institutions, bilateral donors and the governments of the Western Balkans countries. The range of technical assistance includes feasibility studies, as well as economic and financial analyses, environmental and social impact assessments, and design drawings.

Figure 4 Railway lines in Corridor 5-c and BiH Core Transport Network

2 Railway lines Šamac-Sarajevo and Sarajevo-Čapljina

Back in 2004 The World Bank in “Bosnia and Herzegovina Infrastructure and Energy Strategy” assessed that 70% of the railway network in BiH is in need of major rehabilitation. A year later, EBRD recognized improvement of the railway infrastructure as a key priority in the country. Due to poor condition of railway infrastructure and lack of signalling and telecommunications systems maximal train speeds in BiH are limited to 70 km/h for passenger trains and 50 km/h for freight trains.
Railway lines in Corridor 5-c are D4 category (fiche UIC-700) conventional rail system for mixed traffic (yet dominantly used for freight transport) providing standard gauge – GB (fiche UIC-506). Both railway lines are included in “SEETO Comprehensive Network”, so at some point it will also be included in the Trans-European Transport Network as well. However, considering the present condition of infrastructure it seems that some urgent measures should have been applied a long time ago.

2.1 Railway line (km 21+748) – Šamac – Sarajevo (km 257+097)

Construction of railway line Šamac-Sarajevo in the valley of the Bosna River (maximal gradient is 8%) was completed in 1947. The railway line was significantly upgraded from 1968 to 1971 (railway line electrification, installation of automatic block signalling and electric power supply remote control etc.). Construction of the second track from Doboj to Jelina (87 km) was completed in 1978 and applied design parameters should have enabled train running speed from 70 to 100 km/h. The railway line is constructed in densely inhabited areas. Railway stations Sarajevo and Zenica are located in city centres and there are many settlements along most of the alignment. Consequently, one of the big issues is safety (e.g. between Sarajevo and Zenica there are 39 level crossings).

2.2 Railway line Sarajevo – Čapljina – (km 170+390)

Railway line Sarajevo-Čapljina-Ploče was designed and constructed over mountain pass “Ivan sedlo” (narrow track tunnel “Ivan” was reconstructed to provide standard track gauge) and through the valley of the Neretva river. The electrified (25 kV AC, 50 Hz) single track railway line was put in operation in December 1966. The mountain railway line section Bradina-Konjic was designed and constructed with marginal technical standards (i_{max}=25 ‰ and R_{min}=250 m). The applied design standards allow maximum train speeds between 70 km/h (the mountain section) and 100 km/h (southern from Mostar). The design included a lot of railway structures so 98 tunnels as well as 70 bridges, viaducts and overpasses were constructed.

2.3 Railway lines in Corridor 5-c Rehabilitation Projects

2.3.1 "Railways I" Project

"A General Overhaul of the Single Track Railway Line Sarajevo-Čapljina, (30 km long) Section Bradina-Konjic-(Čelebići)" was completed in 2005. Project also included reconstruction of border stations Čapljina and Šamac and railway station Doboj, as well as rehabilitation of stations interlocking and level crossings.

2.3.2 "Railways II" Projects

Construction works on Track overhaul of 100 km long section Čelebići – Čapljina were completed in 2011. Signalling system rehabilitation works started in July 2010 but hasn't been fully completed yet.
Track overhaul of the railways on Corridor Vc section Sarajevo-Bradina including tunnel "Ivan" rehabilitation is on pending (design for 41,5 km long section was completed in 2008 and General Procurement notice was issued on 21st November 2013). Invitation to tenders shall be issued soon.
"Reconstruction of track and signalling works on railway Šamac – Doboj" is on pending as well (design for 62,5 km long section was completed in 2008, but Feasibility Study is still missing). Invitation to tenders for modernization signalling-telecommunication system along Railway lines Banja Luka- Doboj and Sevarlije-Doboj-Samac was issued on 31st January 2014.

2.3.3 WBIF Projects

Technical Assistance for "Track Overhaul of Railway Section Podlugovi-Sarajevo on Corridor Vc" (Feasibility Study and Design for overhaul of 25 km long single track section) was successfully completed in 2013.
Technical Assistance for "Track Overhaul of the Railway Sections Doboj-Maglaj and Jelina-Zenica on Corridor Vc" (Feasibility Study and Design for overhaul of 23,3 km long double track section and 8,8 km long single track section) was successfully completed in 2014.

3 Lessons learned?

One of the main reasons why the railways in BiH are struggling to improve poor performances lies in lacking strategic planning. EU transport policy concepts (e.g. sustainable mobility) and projects (e.g. shift2rail) should be top priorities on the agenda of BiH politicians. The Governments are aware that financial stability of infrastructure manager(s) is a pre-requisite for implementation of priority railway projects, so restructuring of ZFBH and ZRS can be expected rather soon.
Railway lines in Corridor 5-c are not only one of the most important transport systems in the country but also in a wider regional context (e.g. essential for operation of Croatian port of Ploče). Therefore, introduction of track maintenance planning based on Life Cycle Costing and Asset Management System is a must!!!
It seems that the harmonization of railway standards and regulations and transposition of EU legislation is the only bright spot of the railway sector in the country (confirmed with the highest mark in new EBRD Strategy). Still, RRB and BHRPC are not fully operational and there are a lot of railway regulations still pending.
Rehabilitation of railway lines in Corridor 5-c (including signalling, telecommunications and electric power supply facilities and equipment) remain at the top of priorities. However, reha-

bilitation of landslides and decontamination of mine fields prior to railway line rehabilitation works is paramount.
In addition, even on sections where railway line rehabilitation works were recently completed there is a lot of railway structures (tunnels, bridges, viaducts, masts etc.) in a poor condition, therefore a railway asset management system should be introduced as soon as possible.
Modernization of signalling (including level crossings) and telecommunications of Railway Lines in Corridor 5-c is a big issue and it seems the time has come to treat it accordingly. In other words, the consolidated technologies and systems will be applied as described in TSI.

4 Conclusions

The rehabilitation of railway lines in Corridor 5-c is progressing, but the feeling that prevails is that the pace has been too slow. It took ten years to complete (single) track overhaul works between Bradina (km 41+084,54) and the Southern Border with the Republic of Croatia (km 170+390), which is roughly just a third of the total single track length. As there is also an 87 km long double track section, with this pace it would take at least another 20 years to complete the overhaul.
Restructuring of railway companies in the country and full staffing of BHRPC and RRB are prerequisites for cost-effective railway infrastructure maintenance and development of feasible projects as well. Moreover, the BiH governments and railway authorities should make harder efforts to accept the European sustainable mobility concept and draft strategic papers promoting comparative advantages of railway transport in the Comprehensive Transport Network.
Transposed European Norms together with new regulations stipulate the introduction of modern technologies preparing the system for future integration into the Trans-European Transport Network. Finally, as the International Financial Institutions are willing to continue playing their role using reliable resources and tools for funding core transport infrastructure development in the country, the main question is not if but when BiH will have a modern conventional railway system.

References

[1] BHRPC: “Investment Plan for Railway Infrastructure in BiH in the period 2010-2014 with the development prospective up to 2020” (12/2009);

[2] “Stabilisation and Association Agreement between BiH, of the one Part and the EC and their member states, of the other Part” (2008);

[3] DB International: “Study on TER Compliant Railway Corridor Vc in BIH” (2008);

[4] DEI BiH: “EU Integration Strategy of BiH” (2006);

[5] The BiH Railway Act (8/2005);

[6] WB: Bosnia and Herzegovina Infrastructure and Energy Strategy (2004);

[7] JICA for the World Bank: “Study on the Transport Master Plan in BiH” (2000);

[8] OHR: “Agreement between FBiH and RS on the establishment of joint Railway Public Corporation as part of Transportation Corporation” (4/1998);

[9] www.eib.org

[10] www.ebrd.com

[11] www.worldbank.org

[12] www.wbif.eu

[13] ec.europa.eu

[14] www.seetoint.org

RAIL TRAFFIC NOISE PROTECTION IN CROATIA – CHALLENGES DURING THE FIRST APPLICATION

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Abstract

In order to upgrade the track structure for train speed of up to 160 km/h, approximately 62 km long section of railway line Oštarije-Knin-Split is in process of reconstruction. To complete the reconstruction, noise protection and railway level crossings safety features need to be incorporated on the track. This paper presents the process of rail traffic noise protection project creation. Project predicts both active and passive noise protection measures – construction of noise protection walls and building isolation. Noise protection walls described in this paper will be the first rail noise protection structures on Croatian railway network.

Keywords: rail traffic noise, environmental protection, concrete noise wall

1 Introduction

In order to upgrade the track structure for train speed of up to 160 km/h, approximately 62 km long section of Lika railway, i.e. section from station Perušić to Gračac of railway line Oštarije-Knin-Split (shown in Fig. 1) is in process of reconstruction. As final part of the reconstruction project, rail noise protection must be designed. This task has been entrusted to the Department for Transportation Engineering of Faculty of Civil Engineering Zagreb.
According to [1], traffic noise from newly built and reconstructed transport infrastructure such as railways, state or county roads, that adjoin or intersect the residential, business, vacation, recovery and treatment areas should be designed and constructed in such a way that the noise level at the border of planned transport corridor does not exceed the equivalent noise level of 65 dB (A) during the day and 50 dB (A) at night. According to [2], if predicted rail traffic noise levels, calculated by "interim" noise computation method – Dutch National Method RMR-SRM II, should exceed prescribed values, mitigation measures must be carried out.
Commonly used method of rail noise control both on existing and new railway lines is construction of noise walls next to rail tracks, often in combination with building noise insulation. Typical noise reductions after wall installation are up to 10-15 dB [3] depending on the wall height, absorption and its distance to source and receiver. A survey conducted by UIC [4] showed that, by the end of 2005, more than 1.000 km of noise walls were constructed and more than 1.25 million Europeans have noise protection through insulated windows. An estimated total of 150-200 million euros are spent annually in Europe on noise walls and insulated windows. Same measures of noise protection will be used on this section of Lika railway.

Figure 1 Section from station Perušić to Gračac of railway line Oštarije-Knin-Split

Project assignment dictates design of noise protection exclusively for buildings near open rail track sections, while areas around three rail stations (Perušić, Gračac and Gospić, Fig. 1) are excluded from consideration. The project of rail traffic noise protection is divided into three parts:

- noise protection study which contains acoustic modelling of the observed area and results in optimal definition of noise protection wall's height and its placement in rail plan;
- noise protection wall main design which contains elaboration of its elements and positioning in vertical plane and
- proof of proposed protective construction mechanical resistance and stability which contains calculation and dimensioning of noise protection wall elements.

Although the procedure of noise protection design and construction during (re)construction of road infrastructure has become common and well known during the last years, as this is the first case of noise protection application on Croatian railway network, during the project development few challenges emerged. These challenges, as well the procedure of three-part project creation will be briefly presented in the following sections.

2 Creation and results of noise protection study

In order to perform the acoustic analysis and determine the optimized wall height and length, placement of wall in rail track cross section had to be established. To achieve optimal performance of the noise wall, a general rule is to place it as close as possible to the source or to the receiver.
Two challenges emerged while predicting the wall's location in railway cross sections. The first was non-existent national regulations regarding noise wall placement along the tracks i.e. its minimum distance from track axis. Draft version of such a document states that the distance between rail track and wall axis must be greater than 4 meters. This demand, although unofficial, created another problem.
According to project task, walls must be constructed on the rail reserve. The lack of free space between the nearest rail and rail corridor boundary for wall structure placement, i.e. already built drainage elements and telecommunication installations, in many track sections made wall introduction impossible (Fig. 2). After considering specifications defined in Italian and German national regulations, Investor decided to reduce the permissible minimum distance to 3.6 meters (Fig. 2).
Finally, it was time to optimize the planned noise protection. Since the track section is under reconstruction, rail traffic is carried on a reduced scale and with limited train operational speed (Table 1), study predicts two phases of noise protection construction.
Phase I includes the noise protection measures (both passive and active) that must be implemented immediately, even in these exceptional rail traffic conditions. The construction of phase I noise protection walls and building isolation coincides with completion of rail level crossings safety measures incorporation, i.e. marks the end of rail section reconstruction. After planned protection in phase I is constructed, it will be necessary to conduct acoustic field measurements in order to determine the effect of preformed protection measures.

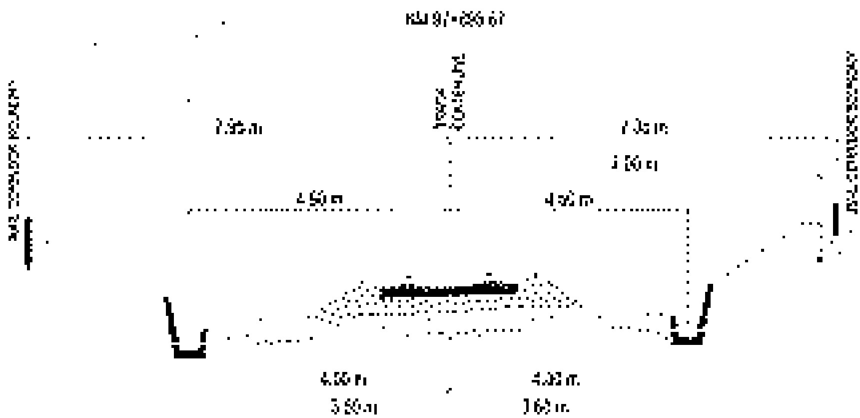

Figure 2 Typical cross-section of Lika railway from Perušić to Gračac

Table 1 Noise modelling input data regarding rail traffic in operation during reconstruction

Train category	Design speed	Max speed	Number of trains during		
			Day	Evening	Night
	[km/h]	[km/h]	07 – 19 h	19 – 23 h	23 – 07 h
Passenger	100	100	5	0	4
Freight	80	80	7	2	3

Because the completion of track reconstruction implies an increase in rail traffic volume and speed (Table 2), the study further predicts the necessity of the implementation of the phase II protection whose construction should start immediately after the completion of phase I.

Table 2 Noise modelling input data regarding rail traffic in operation after reconstruction

Train category	Design speed	Max speed	Number of trains during		
			Day	Evening	Night
	[km/h]	[km/h]	07 – 19 h	19 – 23 h	23 – 07 h
Passenger – tilt	160	110-160	8	0	0
Passenger – conventional	120	80-120	2	2	6
Freight	80	80	8	2	3

Table 3 shows the height, length and area of optimized noise protection walls of protection phase I and II.

Table 3 Dimensions of optimized noise protection walls – phase I and II

Wall height	Phase I		Phase II	
	Lenght	Area	Lenght	Area
H [m]	L [m]	A [m²]	L [m]	A [m²]
1.0	0	0	24	24
1.5	20	30	96	144
2.0	100	200	184	368
2.5	212	530	16	40
3.0	80	240	24	72
3.5	0	0	12	42
Total	412	1000	356	690

According to the investor’s decision, individual residential buildings located at a minimum distance of 150 m from the adjacent buildings are not intended for protection by noise protection walls. Such facilities will be protected by passive noise protection measures. In addition, the study showed that passive protection should be applied on objects close to the rail level crossings. Noise modelling results showed that 19 residential buildings are in need of noise insolation.

3 Creation and results of wall’s main design

While deciding about the type and materials of wall’s construction, factors such as limited space in rail cross section, available construction technology, aging/corrosion, stone impact and fire resistance were taken into account. Therefore, project predicts the construction of the walls with steel columns, supported by concrete piles and fill slabs made out of precast concrete. In most countries, precast concrete is the most commonly used wall material, providing low cost, low maintenance and effective solutions to unwanted noise. This is because concrete noise protection walls [5]:

- are durable, with a design life of at least 40 years;
- require minimal maintenance and low whole life costs;
- can act as safety walls, withstand elements, fires and vandalism;
- take up less space than earth mounds;
- are made from locally produced materials;
- can be designed for installation at a variety of angles – vertical, raked or mixed;
- move from being an exposed structure to acting as embankment stabilizers in a continuous ribbon;
- are flexible in design – can have any profile, colour or size can and therefore provide elements of architectural interest;
- are plant friendly – they contain no preservatives and need no repeat treatment.

4 Creation and results of wall's mechanical resistance and stability calculations

While calculating the mechanical stability and resistance of wall structure the following three effects were observed:

- structure dead load together with additional constant load (weight of the absorbing layer of wall fill slabs) in the value of 1.9 kN/m^2;
- wind pressure in its maximum value of 1.37 kN/m^2 (for construction located in wind zone III and at an altitude of 560 meters above the sea level wind speeds are up to 35 m/s) [6];
- rail vehicle pressure – this is yet another challenge that needed to be solved, because, again, there is no national legislation covering this topic. The answer was found in German researches and practice. According to [7], the maximum value of pressure and suction for train passing speed of 200 km/h is 0.35 kN/m^2. This value is considerably lower than wind pressure. Considering that the maximum train speed on reconstructed section of Lika railway will be 160 km/h and that regulations require that wind and vehicle pressure are not to be taken into calculation simultaneously, wind pressure was defined as the relevant load for wall's mechanical resistance and stability calculations.

Due to the possibility that the results of acoustic measurements preformed after the construction of the phase I walls will show a need to increase the overall height of the walls, all the calculations and dimensioning of construction elements were performed for 0.5 m higher walls. When calculating and dimensioning of piles, data on soil characteristics contained in the geotechnical study [8] was applied. Calculations were done considering the characteristics of the soil that is prevalent in a particular area. This is because the distances between the investigation points described in the study were quite large. However, since it is possible that, in some places, piles will enter the solid rock, two principles of calculating and dimensioning were conducted:

- the pile is hovering in infinite half-space – calculations were conducted by the theories and data of [9, 10, 11, 12];
- the pile is wedged into solid rock in depth of 1.2 meters – calculations were conducted using the finite element method and Winkler soil model.

Calculations showed that the foundation construction of the noise protection walls must consist of piles 0.6 meters in diameter and from 3.5 to 5.0 meters in length. At the top of the pile there should be specially shaped patella for steel pillars in depth of 0.9 or 0.7 meters. The thickness of the supporting layer of reinforced concrete fill slabs must be 0.12 meters. Wall construction is shown in Fig. 3.

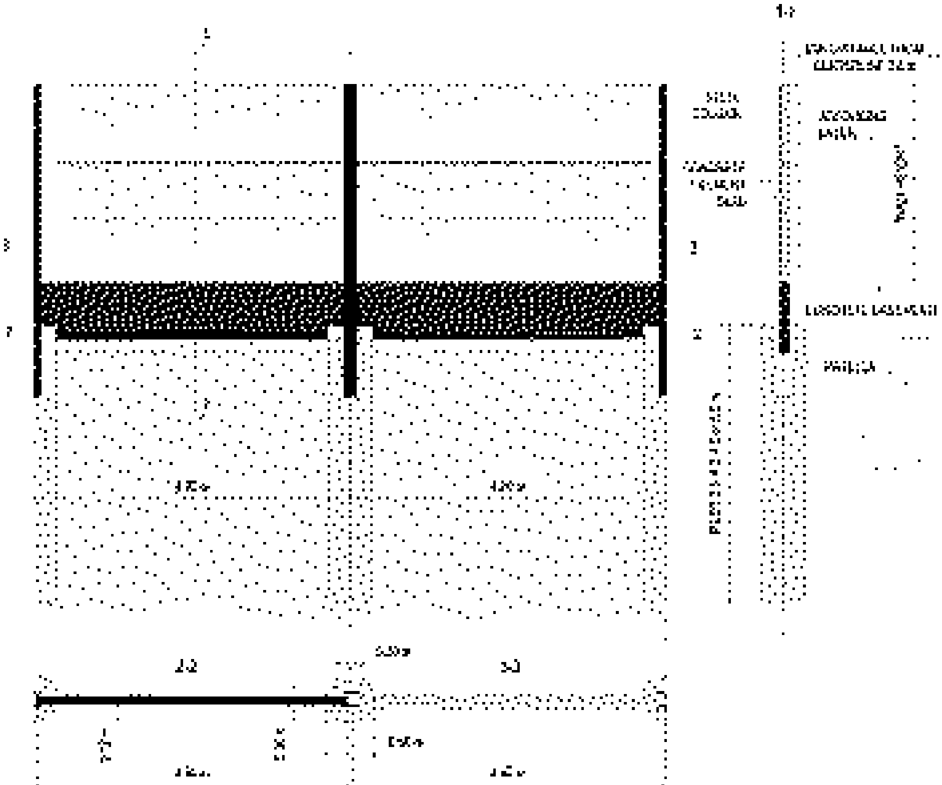

Figure 3 Elements of noise protection wall construction

5 Conclusions

Project of rail traffic noise protection described in this paper had few challenges that were gradually overcame by joint effort of Investor, project manager and designers. The biggest issue to the project designers was relatively late inclusion of noise protection project in the overall rail reconstruction project and lack of legislation to be followed in the design procedure. It is therefore important to emphasize that, for the future projects, noise protection walls design must be incorporated in early stages of the track (re)construction project, as they are an important part of track substructure and not a mere railway equipment.

Based on the existing trends in urbanization, increase in the amount of cargo transportation and environmental requirements, it can be said that the need for high quality railways in Croatia is in constant growth. Such expectations are based on the specifics of railways, long-term investment periods and the significant role of budget funding, especially in the area of infrastructure investments. Due to the economic situation, the planned construction of high-speed railway lines is now delayed and only rehabilitation of railway lines for speeds up to 100 km/h are planned. Nevertheless, this is the gap that needs to be fulfilled with quality noise protection structures construction legislation preparation.

References

[1] Highest permitted noise level regulation for places where people work and reside, Official Journal, NN 145/2004.

[2] Regulation on the noise mapping and content of noise maps and action plans and the calculation of permitted noise indicators. Official Journal, NN 75/2009.

[3] Position Paper on the European strategies and priorities for railway noise abatament, Working Group Railway Noise of the European Comission, Luxembourg: Office for Official Publications of the European Communities, 2003.

[4] Oertli, J. & Hübner, P.: Railway Noise in Europe – A 2010 report on the state of the art, International Union of Railways (UIC), 2010., www.uic.org

[5] Kotzen, B. & English, C.: Environmental Noise Barriers – A Guide to their Acoustic and Visual Design, CRC Press, 2013.

[6] HRN EN 1991-1-4:2012/NA:2012, Eurocode 1: Actions on structures – Part 1-4: General actions – Wind action

[7] Sehorsch, T.: Erfahrungen beim Bau von Lärmschutzsystemen entlang Hochgeschwindigkeitsstrecken, Überwachungsgemeinschaft Gleisbau e.V. Vortragsveranstaltung im Rahmen der Mitgliederversammlung ÜGG, 2. – 3. Sept. 2010.

[8] Railway line Oštarije-Knin-Split, section Perušić-Gračac, Geotechnical investigations, Institute IGH, 2003.

[9] Broms, B. B.: Lateral Resistance of Piles in Cohesive Soils, Journnal of Soil Mechanics and Foundations Division, ASCE, vol. 90, SM2: 27-63., 1964.

[10] Broms, B. B.: Lateral Resistance of Piles in Cohesionless Soils, Journnal of Soil Mechanics and Foundations Division, ASCE, vol. 90, SM3: 123-156., 1964.

[11] Fellenius, B. H.: Basics of Foundation Design, 2nd Expanded Edition, BiTech Publishers Ltd., Richmond, B.C., 1999.

[12] Terzaghi, K.: Evaluation of coefficients of subgrade reaction, Geotechnique, Vol. 5, No. 4, 41-50., 1995.

MAINTENANCE IN THE LIFE CYCLE OF RAILWAY INFRASTRUCTURE

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Abstract

This paper analysis a comparison of the current maintenance of railway tracks with the maintenance based on an analysis of the life cycle costs in order to define a new model for the maintenance of railway infrastructure. The model should generate a minimum total cost in the life cycle of railway tracks for the required level of track quality. The current model of corrective maintenance of track in use was analyzed, and an alternative model of preventive maintenance during the extended life cycle of the track is proposed.
The model was defined using a chosen track section, which was analyzed and reduced to two representative types of standard kilometer. Following these analyses, models for corrective maintenance and preventive maintenance for standard kilometers I and II were obtained, also considering corresponding subtypes depending on the anticipated traffic load up to 1, to 3 and to 5 million gross tons. Benefits for preventive maintenance are lower overall maintenance costs with great effects and longer periods of preserved high quality of the track, while deficiencies are increased maintenance costs at the beginning of the observed period of the life cycle, the need for good preparation of maintenance with strong logistics and unavailability of track for longer periods of time.
Benefits of corrective maintenance are lower initial costs of maintenance due to its use, since they are the only parts of the track where failures were found that are maintained, while the disadvantages are total higher maintenance costs, faster reduction in the overall quality of the track, small effects of maintenance and many short intervals of closed tracks due maintenance with high costs disorders in train traffic. The research results favored preventive maintenance with flexible quality maintenance limits during lifecycle of track.

Keywords: railway network, corrective maintenance, preventive maintenance, life cycle costs

1 Introduction

In the last few decades in Europe significant changes are occurring in the management of railway infrastructure. In all EU member states, but also outside the EU, continuous work on the restructuring of railways is taking place in order to separate the railway infrastructure management and transportation management. Infrastructure Managers (IM) today must ensure the system of railway infrastructure that provides reliable transport of passengers and cargo in a safe manner and that should be as simple to maintain. The decision on choosing the type of railway track is not only determined by the initial investment cost ("cost of acquisition"), but also by the costs that will be generated through the operation and maintenance ("cost of ownership") in the lifetime of the railway track. In order to make this possible, manufacturers and suppliers involved in the project of railway track construction are developing products that are reliable and cost-attractive in order to optimize the cost of acquisition and cost of ownership. This optimization should normally begin immediately at the beginning of the life cycle of the track, during concepts and definitions of the system, in order to anticipate all expenses that will occur during the life cycle of the track. All decisions of the infrastructure manager concerning the design, manufacture and construction of track components have an impact on the performance of the track safety, reliability, maintainability and total life-cycle costs. A key aspect of the railway track, as an essential part of the railway infrastructure, is its long lifespan (30 years or more), because once the track is built, later is extremely expensive and complicated to change its design. This means that decisions taken at the beginning of the life cycle and dynamic decision-making over a period of maintenance have a long-term impact on the lifetime of the track.

2 Maintenance of the railway infrastructure

The life cycle of the railway track system is a time period that encompasses the overall lifespan from the initial concept of the track until its disposal. European norm EN 60300-3-3:2004 Life Cycle Costing [1] defines the life cycle cost of the general system and its application to the track superstructure (Figure 1).

Figure 1 Life cycle of railway track [1]

Due to the long lifespan of the track, operation and maintenance phase is significant for the study of costs. During that phase periodic maintenance activities are performed that generate cost, and its more or less successful effects significantly contribute to the total cost in the lifetime of the railway track. Good maintenance can affect the life cycle of a railway track in such a way as to ensure the expected lifetime of the track, while flaws in maintenance activities can significantly shorten the track lifespan early and cause a irrecoverable decrease quality of the track, need for a subsequent increase in maintenance and therefore higher costs, as may ultimately result in earlier than expected renewal of the track. Rail infrastructure maintenance is a combination of technical and administrative activities, including inspection activities, which aim to preserve the railway track in operation or bring it to a state that can perform the intended function – the circulation of railway traffic in a reliable and secure manner, with a designed capacity and according to the defined requirements for speed limit and the permitted axle load. To design optimal maintenance it is important to ensure the necessary conditions [2] such as: Register of infrastructure as a first step maintenance management system to structure a complete record of railway track and its correlation with quality (determined by measurements), maintenance activities and generated traffic load. These data are mainly in possession of infrastructure manager, but in different databases and forms, which are relatively difficult and complicated for maintenance purposes. Maintenance models that determine methods to monitor the track degradation, frequency of maintenance activities and definition of required maintenance activities. Cost analysis in the life cycle enables the determination of the optimum between investment and maintenance, or comparison and selection maintenance type with the lowest total cost while ensuring the required quality of the track, using developed maintenance models, experiences in maintenance and expert assessment. Planning of maintenance and renewal of the track with the help of computer support facilitates the prioritization of maintenance activities, as well as identifying and planning the necessary resources such as time intervals for maintenance, necessary labor, materials, machinery and equipment for maintenance. Maintenance of the track is classified according to EN 50126-1999 [3] (Figure 2).

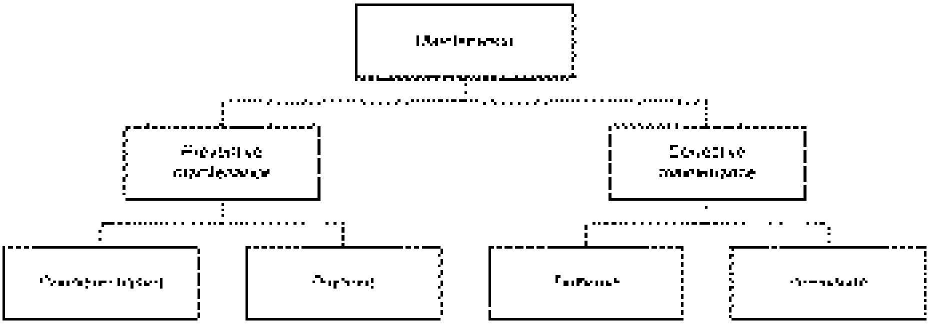

Figure 2 Types of maintenance of railway track [3]

The infrastructure manager during operation and maintenance of the track must make decisions that are aimed at preserving the balance between economic aspects (costs) and safety-technical aspects (safety) of railway track. The aim of the infrastructure manager in terms of maintenance is to find efficient and cost-effective models to optimize the operating hindrances and reduce train speed restrictions which finally contribute to the increased availability of the track in its lifetime.

3 Track maintenance models

On the track in use were considered the costs of present type of maintenance in prolonged life cycle of track and comparison with an alternative way of maintenance while maintaining the required quality of the track. Considered track in operation is single track R202 Dalj-Varazdin, section between Osijek and Nasice 48.159 km long. The section was built in 1974, category of load is D4 (22.5 t/axle, 8 t/m1), maximum speed of trains is 80 km/h. Designed lifespan of this section was 33 years which has been exceeded in 2007. This means the track section has been completely depreciated, so there are considered just projected maintenance costs in its estimated extended life cycle for a period of 12 years (2013.-2024). Superstructure of the considered track is based with wooden sleepers for a length 19.916 km (Table 1) and with concrete sleepers for a length of 28.243 km (Table 2). Based on the overall characteristics of the track section Osijek – Nasice, types of standard kilometers (SK) are defined that best describe the considered track section:

Table 1 Standard kilometer I

R › 600 m		**single track, 38 years old**		
MGT/year	rail/sleeper/track	rail grade	ballast bed	substructure
1 / 3 / 5	49E1, wooden (K), CWR	200-220	crushed stone	good

Table 2 Standard kilometer II

R › 600 m		**single track, 38 years old**		
MGT/year	rail/sleeper/track	rail grade	ballast bed	substructure
1 / 3 / 5	49E1, concrete (K), CWR	200-220	crushed stone	good

For standard kilometers subtypes were developed (a, b, c), depending on the expected traffic load on the track – to 1 (a), to 3 (b) and to 5 (c) millions gross tons per year (MBT). Standard kilometers define the main types of maintenance (work cycles) for track section one kilometer in length. In the following will be presented standard kilometer IIb for traffic load up to 3 MBT and maintenance of the track with concrete sleepers type HZ-70, fastening type K or ZEL, rail type 49E1 in the ballast bed of crushed stone. Track is continuously welded (CWR).

3.1 Corrective maintenance

The present method of track maintenance is mainly based on the principles of deferred corrective maintenance based on track condition and detection of failures that require corrective maintenance. Through review of the recorded data obtained by recording car for track geometry, visual inspection of the track and elements of superstructure, maintenance activities are executed only at places where failures are detected at the limit of intervention (category C) [4] and within the available budget for maintenance. If determined condition of the track is such that endangers traffic safety (exceeds intervention limit), it is necessary to immediately take actions to ensure the safety of railway traffic and take immediate corrective maintenance activities. The costs of such maintenance as compared to the effects achieved are relatively high because, for example, machine tamping performed on the track so far was with a relatively small number of machines and in short intervals between trains, resulting in repeated operating hindrances, long slow orders and trains delayed due to maintenance activities. Unavailability of track during maintenance often means expensive substitution of passenger trains with buses and thus increased maintenance costs, and need for passenger operators to provide additional sets of passenger trains.
A key working cycles with corrective and preventive maintenance are recognized through following maintenance activities: tamping of track (leveling-lining-tamping – LLT), adding crushed stone for ballast bed (necessary after LLT), exchange of sleepers, rail replacement, exchange of synthetic rail pads, small spot repairs and chemical weed control. Taking into account the data on the condition and quality of the track, available databases and available experience in track maintenance enables creation of standard kilometers for corrective maintenance and the traffic load up to 3 MBT/year (Table 3).

Table 3 Standard kilometer IIb – working cycles of corrective maintenance

Standard kilometer IIb															
R>600		single track													
Milions GT/year	rail/sleeper/track	grade	ballast bed			substructure									
up to 3	49E1, concr. (K/ZEL),CWR	220	crushed stone			good									
maintenance	life span	12	1	2	3	4	5	6	7	8	9	10	11	12
renewal	times in life span	0												
leveling-lining-tamping	times in life span	4,1	0,35	0,35	0,35	0,35	0,35	0,35	0,4	0,4	0,4	0,4	0,4	
additional ballast	times in life span	0,18		0,03		0,03		0,04		0,04		0,04		
rail exchange	times in life span	0,03			0,03									
sleeper exchange	times in life span	0												
rail pads exchange	times in life span	0,1		0,1										
small repairs	times in life span	3		1				1				1		
chemical weed control	times in life span	11	1	1	1	1	1	1	1	1	1	1	1	

3.2 Preventive maintenance

An alternate way of track maintenance in the life cycle of railway track is based on planned preventive maintenance for multi-year periods. In this way, maintenance costs of operating hindrances (costs of slow orders, substitutions of passenger trains with buses, delayed trains) are reduced to a minimum in a way to ensure long enough intervals of track possessions for maintenance (duration of min. 6 hours) with numerous and high quality machinery capable for large work effects and maximum utilization of the available time periods for maintenance. That allows execution of maintenance work with smaller total number periods of closed track and overall shorter duration of slow orders with less operating hindrances in railway traffic. Maintenance activities are planned through monitoring of the track quality index (TQIu) [4] in a way to maintain the overall quality of the track at the entire length of track section and up to the maintenance limit and prevention exceeding of intervention limit and need for corrective maintenance. Leveling-lining-tamping activities are executed in approved time intervals of track possessions for maintenance at the entire track section and the quality of the whole length of the track section is increased, as opposed to corrective maintenance where just a parts of the track section are maintained at sections with low quality (Table 4).

Table 4 Standard kilometer IIb– working cycles of preventive maintenance

Standard kilometer IIIb														
R>600		single track												
Milions GT/year	rail/sleeper/track	grade	ballast bed			substructure								
up to 3	49E1, concr. (K/ZEL),CWR	220	crushed stone			good								
maintenance	life span	12	1	2	3	4	5	6	7	8	9	10	11	12
renewal	times in life span	0												
leveling-lining-tamping	times in life span	4	1			1			1			1		
additional ballast	times in life span	0,16	0,04			0,04			0,04			0,04		
rail exchange	times in life span	0,03			0,03									
sleeper exchange	times in life span	0												
rail pads exchange	times in life span	0,1		0,1										
small repairs	times in life span	3		1				1				1		
chemical weed control	times in life span	11	1	1	1	1	1	1	1	1	1	1	1	

3.3 Comparison of maintenance costs

For each working cycle of standard kilometers and for all maintenance activities were calculated maintenance costs per kilometer of track, separately for corrective maintenance and preventive maintenance. Maintenance costs were calculated on the basis of available data of infrastructure manager (IM) HŽ Infrastruktura, Zagreb (period from 2012 and 2013) about costs of maintenance machinery, costs of superstructure materials, costs of labor and costs of operating hindrances. In order to determine the present value of the maintenance costs in expected life cycle of track, maintenance costs are discounted using a discount rate selected at the beginning of the observation period. With summing up the annual costs of the operation and maintenance phase discounted at the beginning of the observation period, present value of maintenance costs are determined in the observed life cycle of a railway track.

Table 5 Present values of corrective maintenance costs (CM) and preventive maintenance costs (PM) for standard kilometer IIb

Standard kilometer IIb													
MGT	concr.	year											
up to 3	total (kn)	1	2	3	4	5	6	7	8	9	10	11	12
Present value (CM)	259004	12123	63460	41849	21074	9974	33901	9881	21039	8962	28611	8129	0
Present value (PM)	228713	38405	41088	30868	33175	3534	12290	28658	3053	2907	32098	2637	0

To compare the costs of current model of corrective maintenance for railway tracks with the proposed model of preventive maintenance in the prolonged life cycle of 12 years, discount factor of 5 % was chosen to calculate the present value of maintenance costs, while the impact of inflation is not taken into account (Table 5). By comparing the value of the cost of corrective maintenance in relation to the expected costs of preventive maintenance for standard kilometer II and traffic load up to 3 MBT, noticeable are the potential savings up to 11.69 % for a given period of prolonged life cycle track in use for a period of 12 years [5]. Maintenance costs for corrective and preventive maintenance in a standard kilometer IIb can be shown in relation to the expected traffic load in the period of extended lifecycle, after which the costs of corrective and preventive maintenance are associated with the corresponding logarithmic regression curve (Figure 3).

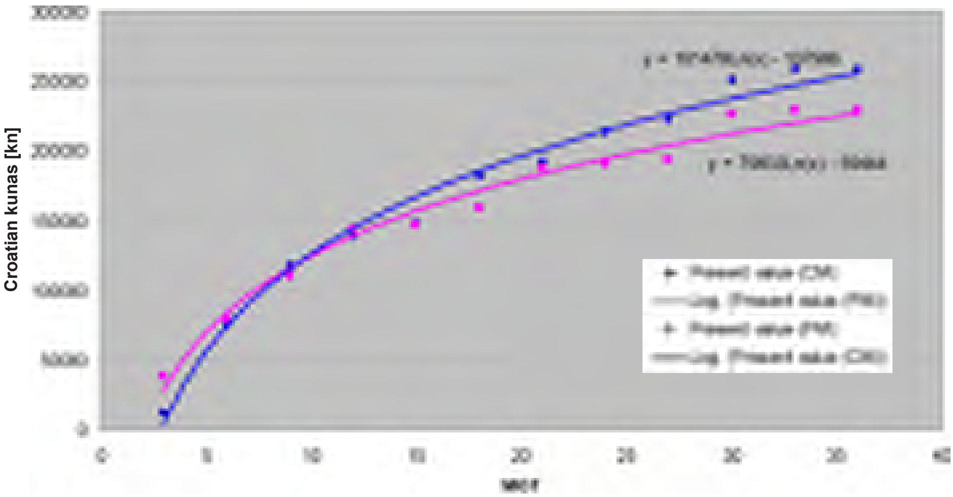

Figure 3 Comparison of corrective and preventive maintenance costs (present values) based on cumulative traffic load for standard kilometer IIb (7,5 kn = 1,0 EUR)

In the case under consideration, the comparison of corrective and preventive maintenance of the standard kilometer IIb shows that the cost of preventive maintenance at the beginning of the observed prolonged life cycle of 12 years are higher than the initial cost of corrective maintenance. Benefits of corrective maintenance are greater flexibility and faster execution of maintenance because it maintains only a portion of the track that requires maintenance, and in the beginning of the life cycle maintenance costs are smaller because of latter start with track maintenance and only on the sections where failures are detected. Disadvantages of corrective maintenance are cumulatively higher costs and higher quality loss than for preventive maintenance, harder prolongation of life cycle without reducing the speed on the track and more intervening corrective maintenance. Advantages of preventive maintenance are longer periods of high quality of the entire track in relation to corrective maintenance, total less maintenance costs, less intervening corrective maintenance because of the high quality of the track and a bigger potential for lifecycle extension of the track. Disadvantages of preventive maintenance are initially higher costs than for corrective maintenance, comprehensive and high-quality preparation to ensure necessary preconditions for the timely performance of maintenance (timely procurement of necessary quantities and types of materials, services, logistics and long enough time periods for maintenance works).

4 Conclusion

Track maintenance should be based on preventive maintenance based on track condition, overall quality of the track, resulting traffic load and past experiences of experts in maintenance. Through monitoring activities such as visual inspection of the track, inspection of track with special equipment for measuring and testing individual elements, inspection of track with locomotive of maintenance personal, analyzing the geometric diagram of the recording car and track quality index provide a basis for planning of preventive maintenance. Through inspections of track failures are observed regardless of their size, but according to their seriousness the necessary emergency corrective maintenance should be performed, and for the next period activities of delayed corrective or preventive maintenance should be planned. The fact is that through planned maintenance activities and working cycles it is not possible to achieve the level of quality that existed immediately after the track renewal [6], but it is also the fact that maintenance activities can slow down decline of track quality and ensure a certain lifecycle extension of the track. Preventive maintenance activities normally should be planned years in advance and for the total expected lifecycle of the track, while the potential number of failures for corrective maintenance activities is estimated based on past data on the frequency of errors and their impact on the traffic disturbances. For a successful preventive maintenance it is required to ensure quality computer support and development of software solutions which will be based on collected detailed data on the condition of track components, data on the changes of the quality of the track during operation in relation to traffic load, data on the quality of completed maintenance activities and data about previous maintenance costs. In that way it will be possible to successfully implement and realize the maintenance of the track based on the lowest whole life costs through planned life cycle of track.

References

[1] EN 60300-3-3:2004 – Dependability management. Application guide. Life cycle costing.

[2] Zoeteman, A.: Life cycle costs for managing railway infrastructure, Delft University of Technology, 2001

[3] EN 50126:1999 Railway applications — The specification and demonstration of Reliability, Availability, Maintainability and Safety (RAMS)

[4] 339a Uputa za provjeru geometrijskoga uporabnog stanja kolosijeka tračničkim mjernim vozilom tehničko mjernih karakteristika EM-120, HŽ Infrastruktura, 2005

[5] Alduk, W.: Održavanje u životnom ciklusu željezničke infrastrukture, magistarski rad, Građevinski fakultet Osijek, Osijek, 2013 (mentor: Marenjak, S.) (in Croatian)

[6] Trameo, Investment and maintenance strategies, Final report, Graz University of Technology, 2006

TRACK GEOMETRY MEASUREMENT AS PREVENTIVE MAINTENANCE DATA SOURCE

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Abstract

Train or metro operating safety may be ensured only if the actual track geometry is known. This requirement includes also knowledge of the horizontal and vertical rail head wear. Analysis of this data is the source of cost effective and rational maintenance decisions. Moreover, the earlier the necessary repair is done, the less expensive it will be, as one may . Therefore, expenditures on measurement equipment are the most effective investment – saving costs of possible accidents and costly repairs if they are made late. This is even more evident in case of track network revitalization, as one has to decide where and what should be repaired first. Any measurement method or tool without the objective data logging feature, makes it virtually impossible to collect all data that would be necessary for the detailed diagnostic reasoning on a line or railway network level, being useful only for direct measurement of some track parameter in a particular point. The principle of track geometry measuring trolley is described that can record all aspects of track alignment and riding quality on plots and tabular reports, so that maintenance service can locate efficiently the specific locations needing maintenance. The measurement results obtained with the trolley – also of the rail head wear – are described along with several track condition assessment methods.

Keywords: track geometry measurement, rail head wear measurement, turnout geometry measurement, track condition assessment, track condition reporting

1 Introduction

Rail transport is one of the safest forms of surface transportation [1], yet railways, metro and trams must carry out regular inspection and maintenance to minimise effect of the continuous infrastructure deterioration which could disrupt freight operations and passenger services. All such work has to be inspected, and the relevant measurement data has to be saved for reference, for checking the infrastructure deterioration rate, or – in the worst case – in the investigation procedures after the eventual accidents.

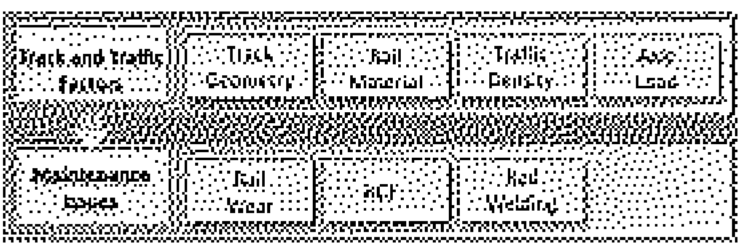

Figure 1 Track maintenance issues

Rail wear, plastic flow, and rolling contact fatigue (RCF) become more and more important issues for railways nowadays. They are the major factors of loss of rail head section and development of surface cracks, depending on conditions like train speed, axle load, rail-wheel materials type, size and profile, track construction, characteristics of bogie type, total load carried (MGT), curvature, traffic type, corrosion, and environmental conditions [2], [3] (Figure 1).

1.1 Crucial role of correctness of rail head/turnout and wheel profiles

The track rails guide the conical, flanged, linked wheels, so the cars stay on track without any active control both in the tangent track and in moderate curves (down to a radius of about 500 m). On sharper curves, the width of the wheel tread is not enough to maintain this effect, and in addition, the wheel flange rests on the high rail side face. If the rail head is side-worn or the flange is worn, then flange climbing is likely and the wheelset may run outside the rail. Tram cars are designed nowadays with low floor levels, so they are the exception as rail cars, as their wheels are not linked, therefore, much benefit in vehicle guidance is lost. Because of that, the tram tracks calls for as much attention as railway lines, even as the line speed is lower. As maintaining the correct rail head/turnout profile is crucial for safety of train operation and extending the track life, it is important to monitor the rail head/turnout profile and to compare it with the desired profiles, usually using the templates specified in the pertinent regulations. Rail profile grinding was initially developed by mining railroads of western Australia to control wear on curves and a dramatic decrease in rail wear was observed with the increase in rail life of up to 80% [4]. Therefore, the profile measurement systems are being needed more and more nowadays to monitor the profile [5]. Efficiency of rail/turnout profile assessment is significantly increased by use of the 'virtual templates' applied to profiles measured by a special trolley [6], or track/turnout recording vehicles [7], [8].
All requirements concerning the correct rail head (also turnout) and wheel profiles, as well as the track geometry quality, become even more complex in case of the Tram-Train concept initially developed in Karlsruhe in Germany, which is now spreading rapidly through Europe [9], [10]. The main idea has been to link a street tram network to the existing local passenger rail network. In this case, the wheel-rail interface is a key to the vehicle's safe operation and controlling factor of the maintenance costs of such system.

1.2 Effect of track geometry on vehicle behaviour in the track

On curves, the outer rail is usually at a higher level than the inner rail (except the tram track), unless the line speed is very low, of max about 15 km/h. However, a particular superelevation may be most effective for a limited range of speeds only. Speed reduction below the lower value of the speed range for the particular superelevation results in the excessive wear of the lower rail head, so speed reductions in a poor quality track may even make its deterioration faster.
Yet another important factor affecting safety of rolling stock operation is the track twist. This parameter should be carefully monitored as it may be a cause of the car wheel unloading, which may lead to derailment. Such risk appears if the track cant varies considerably over the vehicle wheelbase. This is effect is especially detrimental for vehicles with the suspension stiff in torsion. In addition, if cant irregularity is cyclic and its resultant effect on trains travelling at a particular line speed corresponds to the natural frequency of some vehicles passing there, there is a risk of resonant harmonic oscillation – cyclic roll – in these vehicles, which may lead to their extremely improper movement.
Similarly, cyclic errors caused by changing vertical track unevenness can result in vehicles lifting off the track, especially of the cars whose suspension is too stiff when they are empty. In such case, the vehicle wheelsets become momentarily unloaded vertically, so the flanges do not fulfil their function properly, or wheel tread contact is inadequate. It may also happen that the track may be simply grossly distorted because of earthworks.

1.3 Main causes of accidents

Statistical analyses examining the accident cause, type of track and derailment speed [11] revealed that the main cause of derailments were broken rails or welds. Track geometry, excluding wide gauge, was the second cause of derailments on main lines – Table 1. However, for the main track, wide gauge was also one of the most important causes of accidents (3.9%).

Table 1 Main causes of accidents (based on [11])

Main track		**Siding track**		**Yard track**	
Cause	Frequency	Cause	Frequency	Cause	Frequency
Broken rails or welds	15.3%	Broken rails or welds	16.5%	Broken rails or welds	16.4%
Track geometry	7.3%	Wide gauge	14.2%	Wide gauge	13.5%

Train operation safety related issues require, therefore, measuring equipment for rail head profile and track geometry, making it possible to enter and save annotations with visual inspection results (e.g., broken rails or welds), addressing this way the most important causes of accidents [12], [13].

2 Design requirements for track geometry measurement devices

All issues discussed above have to be addressed with the relevant measurement devices, which will provide objective data needed for diagnostics of track and turnouts geometry. Measurement of wheel geometry is not discussed here, albeit the approach demonstrated in this paper makes it possible to take wheel profile into consideration too. The decision was made to develop equipment to be used by maintenance services, acting either based on the information obtained from the periodic track and turnouts measurements of main lines made with the track recording vehicles e.g. [7], or checking the side and yard track network. Therefore, the general design requirements were selected as follows:

- Hardware features
  - portable
  - ruggedized
  - resistant to adverse climatic conditions
  - no calibration required before the measurement session
  - easy to assemble and disassemble
  - long operating time, best on hot swappable batteries
- Control system
  - custom made, to be independent from closed architecture of commercially available products
  - immediate access to functions needed most often
  - capability to work at night – backlit display
  - collection of visual inspection results has to be possible
  - capability to collect and store measurement data for a number of measurement sessions
  - measurement data file has to contain metadata describing its contents, measurement date and time, operator data, and device ID
  - collected measurement data has to be tamper-proof, protection from any corrections to measurement results has to be ensured
  - measurement data has to be transferred to the PC for analysis and archiving
- Software
  - measurement data from different devices, providing the same type of information (e.g., track geometry, rail head profile, et.) has to be processed by common software system
  - the user has to be able to specify tolerances
  - routine analysis tasks have to be done using predefined functions

3 Devices for track and turnout geometry measurement

According to maintenance issues discussed in Section 1 the relevant devices were designed meeting design requirements put forth in Section 2. Two field tested devices are presented below:

- trolley for track geometry and rail head profile measurement – TEP [6]
- optical system for rail and turnouts profile measurement – Scorpio [8]

3.1 Track geometry and rail head profile trolley

The trolley (Figure 2) can measure track gauge, cant, horizontal and vertical unevenness, twist, gradient, distance and rail head profile (non-contact laser vision system), including grooved rails. It is possible to merge measurement data files to represent longer track segments than measured in a single measurement session. This way, obtaining a complete information on the track condition is possible as it contains – its geometry data along with the rail head vertical- and horizontal wear.

Figure 2 Track geometry and rail head profile trolley

Test measurements shown in Figure 3 were taken in the tram track on the sharp curve. They revealed that rail head profile data can be collected 15 times faster with the trolley than with some other laser based reference device. Moreover, the developed trolley guarantees synchronisation of rail head profile with track geometry data. As regards rail head profile data processing (Figure 4), it was 30 times faster, and moreover, retains information about location of the profiles.

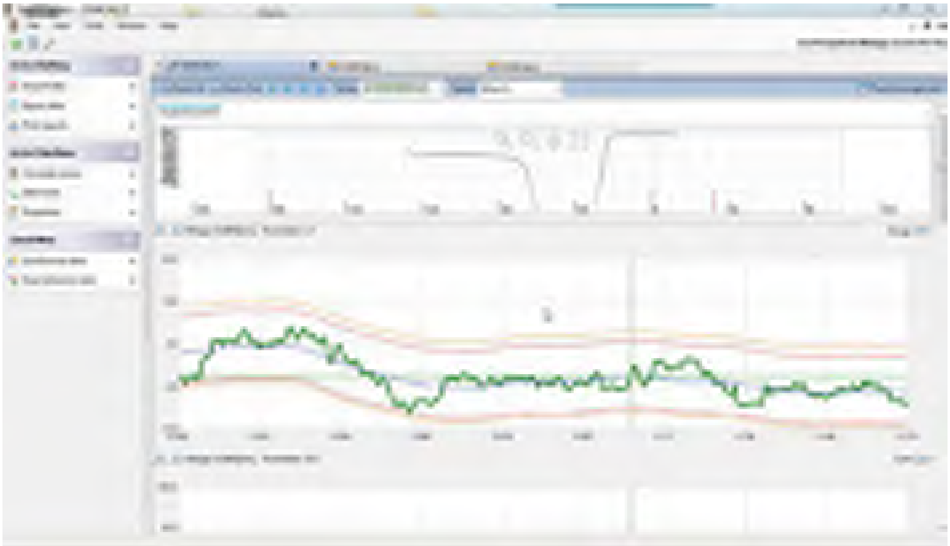

Figure 3 Exemplary track geometry and rail head measurement results

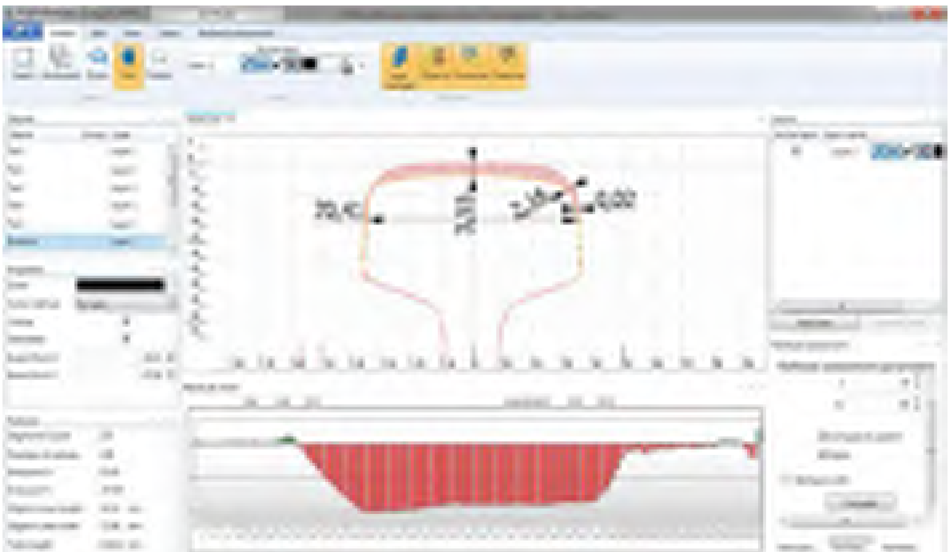

Figure 4 Rail head wear processing example

Several track geometry readings can be overlaid to reveal track deterioration proceeding faster at particular locations. Information about the track geometry is presented along with the information about the defects and events in the track.

Rail head profile can be analysed to support rail grinding technology, providing information about grinding indices, and analyses for the low and high rails separately in the curves. Evaluation and presentation of a number of track condition assessment results is available, e.g. distribution of tolerance exceedings – Figure 5.

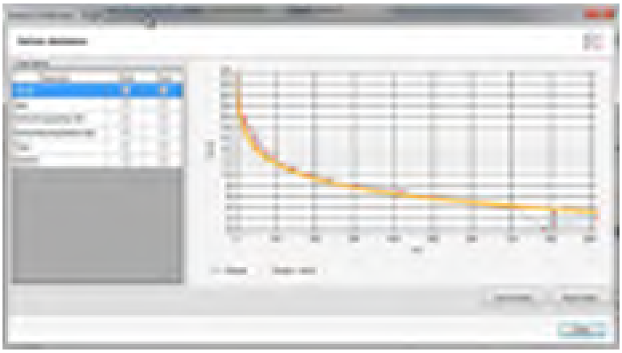

Figure 5 Exemplary analysis of distribution of defect types in the track

3.2 Optical rail and turnouts profile measurement system

The device is composed from the rigid datum frame, movable laser measurement head and its drive system. The measurement result is the 3D model of the measured object (e.g. turnout), being the exact representation of the measured object both in the lateral and transverse directions. Merging of multiple measurements into one object, generating the arbitrarily selected 2D profiles, calculation of the longitudinal profiles, and generating measurement reports.

Figure 6 Optical system for track and turnout measurement

4 Conclusions

Presented devices were designed to collect track and turnouts geometry data, as required for ensuring train operation safety. Both devices were field tested, and verified using independent measurement methods, outperforming them.

References

[1] U.S. Bureau of Transportation Statistics (2010).National Transportation Statistics. Table 2-1: Transportation Fatalities by Mode (Report). http://www.rita.dot.gov/bts/sites/rita.dot. gov.bts/files/publications/national_ transportation_statistics/html/table _02_01.html_mfd – (January 2014)

[2] Reddy, V.: Modelling and Analysis of Rail Grinding & Lubrication Strategies for Controlling Rolling Contact Fatigue (RCF) and Rail Wear, Queensland University of Technology, Brisbane, Australia, 2004

[3] Chattopadhyay, G., Kumar, S., Larsson-Kråik, P.O. and Kumar, U.: (2006) Estimation of parameter for degradation of rails, Proceedings of 19th International Congress on Condition Monitoring and Diagnostic Engineering Management, (Comadem – 2006), Luleå, SWEDEN, 12th -15th June, 2006, pp. 605 – 612,

[4] Nath, D., Ashok, K.: Rail Grinding Necessity on Indian Railway, Course No:625 Sr Professional (P Way), http://wiki.iricen.gov.in/doku/lib/exe/fetch.php?media=625:3rail_ grinding.pdf, 2005

[5] Sroba, P., Roney M.: Rail Grinding Best Practices, Proceedings of the Annual Conference of the American Railway Engineering and Maintenance of Way Association, Chicago, IL, September, 2003

[6] Track Geometry and Rail Head Profile Trolley. User Manual, GRAW, 2013

[7] Madejski, J.: TMS – turnout & track measurement system, Railway Engineering 2009 – 10th International Conference & Exhibition, University of Westminster, London, UK, 24-25 June 2009 , pp. 47-52

[8] Scorpio – Optical Rail and Turnouts Profile Measurement System. User Manual, GRAW, 2013

[9] Rail researchers work on UK's first Tram-Train scheme, http://www.hud.ac.uk/news/ /october/railresearchersworkonuksfirsttram-trainscheme.php, University of Huddersfield, 1st Oct 2013

[10] Crosbee, D.: Introducing the UK's first Tram-Train, http://www.railway-technology. com/features/featuretram-train-project-sheffield-irr-network-rail-4149853/ University of Huddersfield's Institute of Railway Research, 9th January 2014

[11] Liu, X., Rapik, M., Barkan, C.: Analysis of Causes of Major Train Derailment and Their Effect on Accident Rates, http://ict.illinois.edu/railroad/CEE/pdf/Journal% /2012/Liu%20 et%20al% 202012.pdf

[12] Saurabh, K.: Study of Rail Breaks: Associated Risks and Maintenance Strategies, Luleå University of Technology, http://epubl.ltu.se/1402-1536/2006/07/LTU-TR-0607-SE.pdf, 2006

[13] Palese, J., Euston, T., Zarembski, A.: Use of Profile Indices for Quality Control of Grinding, http://www.arema.org/files/library/2004_Conference_Proceedings/00060.pdf, 2004

RAILWAY INVESTMENT PLANNING USING DYNAMIC PRIORITIES

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Abstract

Making a company investment plan is a complex and difficult management issue. Project selection and ranking are crucial steps for the company's success in the market. Strategic decisions, such as the development of a project investment plan, depend on many factors, with different relevance usually changeable over time. Some relationships of the system's elements are functions of time. The subject of this paper is time dependent decision making in transport project management. Several transport projects have been evaluated using Dynamic Priorities in Multi-criteria decision making. Here we consider rail investment projects as part of the Serbian railway network.

Keywords: project management, rail projects' evaluation, dynamic priorities

1 Introduction

Making a company investment plan is a complex and difficult management task [1, 2, 3, 4]. Transport projects evaluation has been very often a crucial step in the transport company's success in the market. The decision making in investment planning is a very complicated process because of many relevant factors, such as: stakeholders (owners, regulators, market, politicians…), system boundaries, transparency, and heterogeneous criteria [5].
The author of [4] in his PhD Thesis emphasized that the modeling process should be seen as a creative, dynamic and cyclic process. Decision makers are confronted with the difficult problem of evaluating and choosing among various alternatives of transport projects. Traditionally, Cost-Benefit analysis, CBA, is used as a support in the decision making process in transport projects planning. However, multi-criteria decision analysis has been promoted to be used in the transport sector to overcome some of the shortcomings of CBA [1, 2, 3, 4]. In this paper authors also suggest the usage of the multi-criteria decision making approaches for transport projects' evaluation.
Good decisions depend on the conditions in the future, and conditions vary over time, so making good decisions require evaluation of what is more likely, or what is more desirable during different time periods. This is the crucial reason for using the dynamic approach in the decision making process. Dynamic judgments and the dynamic approach to transport projects' evaluation is the topic of this paper. Basic assumptions of the applied mathematical computation are defined in the book [6]. There are situations in which changes occur in the structure of the problem, some new criteria can be added or old ones removed. Sometimes the judgments about the criteria change but the criteria remain the same. There are still others in which the judgments about the criteria remain the same, but the judgments about alternatives change over time. Finally, all these combinations are possible in practice [6]. The authors of [7] analyzed risks in megaprojects, after defining all relevant risks the special

focus was on the dynamic relations in the model. The Systems dynamics methodology was explained as a tool for better modeling and analyzing the behavior of complex systems. This approach has been used by researchers and project managers to understand various social, economic and environmental systems in a holistic view.
The authors of [8] suggested a new multicriteria analysis approach, with the following characteristics: unification of the criteria, differentiation of the project's performance over time, as a dynamic variable, and a new approach for the transformation of the physical scales to artificial ones. The application of the proposed approach is demonstrated on the example of the transportation infrastructure investments.
The aim of this paper is time dependent decision making in transport project management. Numerous transport projects have been evaluated using Dynamic Priorities in Multi-criteria decision making. This paper is organized as follows. After the Introduction, the second section, named applied methodology, contains basic assumptions of Analytical Hierarchy Process (AHP). The third section, the model for railway investment planning, presents the developed model with all system elements and their mutual relations. Results and discussion is the topic of the fourth section. Finally, the last section is dedicated to concluding remarks.

2 Applied methodology

Analytical Hierarchy Process (AHP, developed by Thomas Saaty) is one of the most popular approaches for Multi-criteria decision making. It is used in the analysis of decision-making and decision-making to solve complex problems whose elements are the objectives, criteria, sub-criteria and alternatives. AHP is one of the very popular approaches and because its ability to identify and analyze the inconsistency of the decision makers in the process of decompression and evaluation of the elements of the hierarchy. AHP in some way mitigates this problem by measuring the level of inconsistency and informs decision-makers about that. This is a static approach, which used fundamental Saaty scale to represent priorities. Expending the AHP approach it's possible to cope with the time – dependent priorities. This new approach is called Dynamic Hierarchy Process (DHP) [9]. Time-dependent decision-making, i.e. dynamic decision-making is something that is often necessary. But these alternatives may evolve over time, together with our preference for them, such as, for example, the actions of the stock market whose prices are constantly changing over time. Dynamic decision making is a reality, not a complicated concept that can be ignored. It is necessary for the technical design problems in which the effects of several project factors change over time and must make the compromise between them, to allow the system to react differently and continuously over his work time. A typical form of the matrix in a dynamic form:

$$A(t) = \begin{bmatrix} a_{11}(t) & a_{12}(t) & \cdots & a_{1n}(t) \\ a_{21}(t) & a_{22}(t) & \cdots & a_{2n}(t) \\ \vdots & \vdots & & \vdots \\ a_{n1}(t) & a_{n2}(t) & \cdots & a_{nn}(t) \end{bmatrix} \tag{1}$$

$a_{ij}>0, a_{ji}(t)=a_{ij}^{-1}(t)$ in discrete situation, when A(t) is consistent, we have $a_{ij}(t)=w_i(t)/w_j(t)$.
It is preferable that you first obtain weights for different time moments numerically solving the entire problem, and then these values approximate time-dependent curves. All relevant equations are developed in [9].

3 The model for railway investment planning

Developed model has the network structure, including four clusters (Figure 1). Each cluster has a certain number of elements, which is explained in this section in detailes. Considered sections (Rail Corridor X in Serbia) are presented in Table 1 [1, 2 , 3].

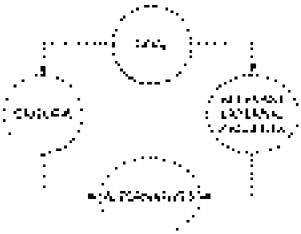

Figure 1 Considered model

Table 1 The considered alternatives with the lengths

Section	Alternative	Length of section [km]	Number of tracks
A_1	Šid-Stara Pazova	116	Double-track
A_2	Subotica-Stara Pazova	153	Single-track
A_3	Resnik-Mladenovac-Velika Plana	70	Single-track
A_4	Velika Plana-Stalać	86	Double-track
A_5	Stalać-Đunis	17	Single-track
A_6	Đunis-Trupale	40	Double-track
A_7	Niš-Preševo	173	Single-track
A_8	Niš-Dimitrovgrad	104	Single-track

All projects are already a part of the "Strategy for the development of railway, road, water, air and intermodal transport in the Republic of Serbia from 2008 to 2015". The purpose of the model in this paper is to rank rail investment projects, considering the financial and operating aspects: C_1 – Cost-benefit ratio, C_2 – Criteria of speed restriction, C_3 – Criteria of rail infrastructure capacity utilization, C_4 – Criteria of inconsistency with AGC & AGTC and C_5 – Criteria of traffic volume [2]. Based on the explanation [2], all defined criteria for considered sections are calculated. The values are given in Table 2. Using these data the pair-wise comparison matrices are developed. The main idea is that alternative with which it can be achieved higher effect is better ranked. The matrix of a criteria comparison is made by expert's recommendation.

Table 2 The calculated values of considered criteria for all alternatives

Alternative	C_1 [%]	C_2 [train hours/km]	C_3 [%]	C_4 [%]	C_5 [train/day]
A_1	0.47	93	17/17	0.014	25
A_2	0.45	24	75	0.084	48
A_3	0.29	333	61	0.192	52
A_4	0.07	31	23/23	0.048	40
A_5	0.08	14	45	0.100	44
A_6	0.05	14	23/23	0.010	44
A_7	1.66	9	34	0.285	23
A_8	2.84	12	36	0.267	16

The relevant external projects [1, 3] can be national or domestic, infrastructure, ecological or social projects, etc. These projects has high importance in the model of transport projects evaluation, having in mind that the transport network is very dependent of its surrounding (including the transport system in the considered country but also neighboring countries). Choosing the relevant external projects should be done by company management or by experts. Suggested relevant external projects in this model are: X – Vidin-Calafat Bridge, Y – Rehabilitation of Corridor IV and Z – Privatization of Port “Bar”. Project X will take flows of goods and passengers from Corridor X and make better service quality on Corridor IV. With project Y the competitive Corridor IV becomes stronger comparing to Corridor X, and with the aim to keep the same freight volume transport on Corridor X, the service quality should be improved. Project Z would increase the volume of freight transport from Montenegro, through Serbia, to Hungary.
We suggested using the dynamic approach for defining the priorities of relevant external projects. Here is explanation for this proposal. One relevant external project has been already realized, Vidin-Calafat Bridge, project X (opened 14. June 2013). The end of the second project, the rehabilitation of Corridor IV is planned for 2020. Project Z, privatization of Port “Bar” will be realized very soon, but its effects are going to be visible in next few years. We assume that relevant time horizon for consideration is from 2014 to 2020 (at that moment all named external projects will be finished).

4 Results and discussion

The model has two parts, static and dynamic. Static part means comparing projects by criteria. This is done by using the Super Decisions software. The dynamic part presents projects evaluation by the relevant external projects. Mathematical computation for this part is done in Matlab. Table 3 presents the alternatives’ weights relative to the criteria. Calculations are done in the software by using the well-known equations from the AHP approach.
The authors assumed that relative priorities of relevant external projects are time-dependent values. Here are pair-wise comparison matrices for relevant external projects, Table 4. Based on the data from Table 4, the functions for priorities are defined (Table 5).

Table 3 Alternatives’ weights relative to criteria

C_i	C_1	C_2	C_3	C_4	C_5	**Weights**
A_i	0.256	0.445	0.182	0.041	0.076	
A_1	0.051	0.211	0.314	0.022	0.037	0.168
A_2	0.237	0.113	0.132	0.044	0.097	0.144
A_3	0.237	0.045	0.063	0.022	0.151	0.105
A_4	0.051	0.075	0.021	0.083	0.220	0.070
A_5	0.237	0.045	0.063	0.083	0.151	0.107
A_6	0.026	0.028	0.314	0.286	0.037	0.091
A_7	0.019	0.045	0.063	0.286	0.020	0.050
A_8	0.142	0.437	0.030	0.173	0.287	0.265
Sum	1	1	1	1	1	1

Table 4 Pair-wise matrices for relevant external projects

	2014			2017			2020		
	X	Y	Z	X	Y	Z	X	Y	Z
X	1	3	5	1	1	4	1	1	1
Y	0.333	1	4	1	1	3	1	1	1
Z	0.2	0.25	1	0.25	0.333	1	1	1	1

Table 5 Dynamic priorities for relevant external projects

	X	Y	Z
X	1	2.875–0.643t	5.333–0.067t
Y		1	4.167t–0.5t
Z			1

According the data from the table 5:

$$a_{12} = 2.857 - 0.643t \quad (R^2 = 0.964) \tag{2}$$

$$a_{13} = 5.333 - 0.067t \quad (R^2 = 0.92) \tag{3}$$

$$a_{23} = 4.167 - 0.5t \quad (R^2 = 0.964) \tag{4}$$

where:

R^2 the R squared values, indicates how well data points fit a statistical model;
t time horizon: 2014 (t=1), 2017 (t=2) and 2020 (t=3).

Thereafter, using the following equations [9], the weights for relevant external projects can be obtained. This calculation is made in Matlab.

$$\lambda_{max} = (a_{13} / a_{12}a_{23})^{1/3} + (a_{12}a_{23} / a_{13})^{1/3} + 1 \tag{5}$$

$$\Delta = a_{12}a_{23} + a_{13}(\lambda_{max} - 1) \tag{6}$$

$$D = a_{12}a_{23} + a_{13}(\lambda_{max} - 1) + (\lambda_{max} - 1)a_{23} + (a_{13} / a_{12}) - 1 + (1 - \lambda_{max})^2 \tag{7}$$

$$w_{e_X} = \frac{\Delta}{D} \tag{8}$$

$$w_{e_Y} = \frac{(\lambda_{max} - 1)a_{23} + (a_{13} / a_{12})}{D} \tag{9}$$

$$w_{e_Z} = \frac{-1 + (1 - \lambda_{max})^2}{D} \tag{10}$$

where:

$w_{ext}, w_{eyt}, w_{ezt}$ are weights of relevant external projects, for X,Y,Z, $t = \overline{1,3}$

The following graphs (Figure 2) are made by using the Matlab.

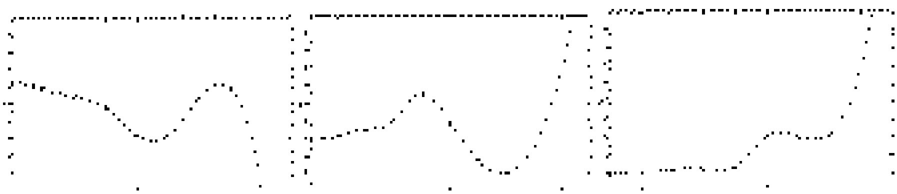

Figure 2 Weights w_{ext}, w_{eyt} and w_{ezt} for relevant external projects X, Y and Z

The weights of alternatives relative to criteria, w_{cj}, are constant value (Table 3), but the weights of alternatives relative to external projects, w_{emt}, are time dependent. We assume that the criterion has the weight 0.7, and the relevant external projects 0.3 for the whole model. Final weights of alternatives in the model should be calculated by following equation (Table 6):

$$w_{A_i t} = 0.7 w_{c_j} + 0.3 w_{e_m t} \quad , \text{ for } \; i = \overline{1,8}, \; t = \overline{1,3}, \; j = \overline{1,5} \tag{11}$$

Table 6 Alternatives' weights relative to external projects and final obtained results of the whole model through time horizons

	2014		2017		2020	
	w_{em1}	w_{ai1}	w_{em2}	w_{ai2}	w_{em3}	w_{ai3}
A_1	0.164	0.168	0.169	0.168	0.200	0.178
A_2	0.145	0.144	0.144	0.144	0.133	0.141
A_3	0.145	0.117	0.144	0.117	0.133	0.113
A_4	0.145	0.092	0.144	0.092	0.133	0.089
A_5	0.145	0.118	0.144	0.118	0.133	0.115
A_6	0.145	0.107	0.144	0.107	0.133	0.104
A_7	0.055	0.052	0.056	0.052	0.067	0.055
A_8	0.055	0.202	0.056	0.202	0.067	0.206

The final alternatives' rank is presented in the Figure 3. The main conclusion is that the relative importance of the alternatives is not changeable through the time, in this case study. However, during the time, some alternatives become more (A_1, A_7 and A_8) or less (A_2, A_3, A_4, A_5 and A_6) dominant.

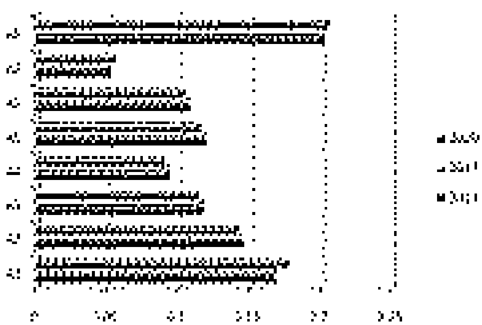

Figure 3 Final alternatives weights through time horizons

5 Conclusions

The model for decision making as a support system in decision making process should have flexible structure, to be easy to change the model according the changes in its surrounding. Decision makers sometimes need a model which take into account the changes of system's elements or changes in system's surrounding. This model presents the results with all possible modifications and gives the suggestions for all of them. The developed model takes into account changes in system's surrounding, with consideration of the relevant external projects, but also includes the changes of elements' priorities over time horizons, giving the final alternatives weights through time horizons. The analyzed case study was Rail Corridor 10 in Serbia. The main conclusion is that the relative importance of the alternatives in this model is not changeable through the time. However, during the time, some alternatives become more or less dominant. With respects to future studies, we recommend conducting the relevant stakeholders and their influences on the decision making process. Very often, there are dynamic stakeholders' preferences, changeable over time.

References

[1] Macura, D., Bojović, N., Nuhodžić, R., Šelmić, M., Bošković, B.: Evaluation of transport projects using multi-criteria decision making method, International Conference on Traffic and Transport Engineering ICTTTE, Belgrade, November 29th to 30th, 2012.

[2] Macura, D., Bošković, B., Bojović, N., Milenković, M.: A model for prioritization of rail infrastructure projects using ANP, International Journal of Transport Economics, Vol. XXXVIII, No. 3, pp. 265-289, 2011.

[3] Macura, D., Nuhodžić, R., Bojović, N., Knežević, N.: One model for rail infrastructure projects selection, 2nd International Conference on Road and Rail Infrastructure - CETRA 2012, Dubrovnik, 7–9 May 2012, pp. 533-538, 2012.

[4] Vestergaard Jensen, A.: Appraisal of Transport Projects – Assessing Robustness in Decision Making, Ph.D. Thesis, Technical University of Denmark, Department of Transport, 2012.

[5] Jowitt, P.: Decisions, decisions, Civil Engineering and Environmental Systems, Vol. 30, pp. 285-293, 2013.

[6] Saaty, T.: Fundamentals of Decision Making and Priority Theory with the Analytic Hierarchy Process, RWS Publications, 2007.

[7] Boateng, P., Chen, Z., Ogunlana, S., Ikediashi, D.: A system dynamics approach to risks description in megaprojects development, Organization, Technology and Management in Construction, 4(3), pp. 593-603, 2012.

[8] Tsamboulas, D., Yiotis, G., Mikroudis, G.: A method for multi-criteria analysis in transportation infrastructure investments, International Journal of Transport Economics, vol. XXXIV, no. 1, 2007.

[9] Saaty, T.: Time dependent decision-making: dynamic priorities in the AHP/ANP: Generalizing from points to functions and from real to complex variables, Mathematical and Computer Modelling, pp. 860-891, 2007.

EUROPEAN EXISTING RAILWAY TRACKS: OVERVIEW OF TYPICAL PROBLEMS AND CHALLENGES

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Abstract

Railways are one of the key modes of transport. The existing aging ballasted tracks play a critical role in rail transport infrastructure. In order to identify the challenges and problems faced by railway infrastructure managers in European countries, a survey was conducted as part of the EU FP7 SMART RAIL project.. The survey consisted of two parts: questionnaires were sent to relevant specialists in various European countries; and a literature survey was undertaken. Key findings include that: (a) rail transport are increasingly globally; (b) with technological advances, rail trends are for faster, heavier and longer trains, as well as increased energy effectiveness and greater reliability, whilst trying to reduce costs; (c) most of the railway tracks in Europe have reached or exceeded their design service life; (d) ballasted tracks are the most popular and economical track bed structure. One of the main challenges is how to enhance these existing ballasted tracks in the context of the trends for increased use. Most countries have their own standards in railway engineering and maintenance. Technical data were obtained from seven countries, which should be useful for engineering design or evaluation. The survey can be used as guidance or/and reference on railway track management and research in particularly in European countries.

Keywords: existing railways, survey, aging, design service life

1 Introduction

Railways are one of the key modes of transport. Aged ballasted tracks play a critical role in European rail transport infrastructure. To identify the challenges and problems faced by railway infrastructure managers in Europe, a survey was conducted as part of the EU FP7 SMART RAIL project [1]. The survey used two means of data collection: questionnaires were sent to relevant specialists in various European countries; and a literature survey was undertaken. The questionnaire was sent to all partners of this EU project SMARTRAIL and some network experts outside the project through email. More than 50 experts were asked to answer the questionnaire. These experts were from 14 countries in Europe: Austria, Croatia, Czech, Germany, Greece, Ireland, Latvia, Poland, Russia, Sweden, Slovenia, Switzerland, The Netherlands, and the United Kingdom. Eight responses to the questionnaire from seven different countries were received, although in most cases the answer to at least one question was omitted. The seven countries were: Croatia, Greece, Ireland, Russia, Slovenia, Switzerland and the United Kingdom (UK).. These are referred to as the respondent countries (RCs). The questionnaire survey mainly focused on track issues, including general issues, vehicles, rails, fastening systems, sleepers, ballasts and sub-ballasts, placed soil and geosynthetics, subgrade, dra-

inage systems, maintenance and the like. The literature survey included the railway market, climate, train speeds and lengths, standards and technologies.
This paper presents the results of the questionnaire and literature survey, followed by a discussion of factors contributing to the European existing railway tracks and future demands.

2 Factors affecting tracks performance

A railway network is a complex system. system is quite a complicated system. First, operations and infrastructure influence each other in various ways. The quality of the rolling stock influences the wear of the infrastructure and thus the amount of maintenance and renewal needed. Secondly, infrastructure failures, but also planned interventions for maintenance, influence the reliability of the operations. An important aspect of rail infrastructure is that the assets or railways system components have long life spans and once installed it is very costly and complicated to modify the initial design. Decisions in design and maintenance will thus have a long-lasting impact. [2]
There are a number of factors affecting the performance of existing railway tracks. Their effects are complex and often inter-connected. Figure 1 shows an Ishikawa diagram (fishbone diagram) to present the key factors that contribute to the behaviour of existing European railway tracks.

Figure 1 Ishikawa diagram (or fishbone diagram) showing the key factors contributing to European existing railway tracks.

3 Results of the survey process

3.1 Methodology of the survey

Figure 2 shows the methodology for the survey process. Two methods were applied to collect data: a questionnaire and a literature survey, [2].

3.2 Results of the questionnaire survey

The age of the railway tracks currently operated in the RC's is presented in Figure 3. It is possible that this question may have been interpreted differently by some respondents. Irish and Swiss responses, who indicate their tracks are relatively new were related purely to railway tracks rather than railway lines. In the case of others, they might have understood that railway lines or track systems were the focus of the question. The answer to the question "what is the oldest railway track system still in service" was much more consistent. Figure 4 shows that many track systems built in the first half of 19^{th} century, are still in operation.

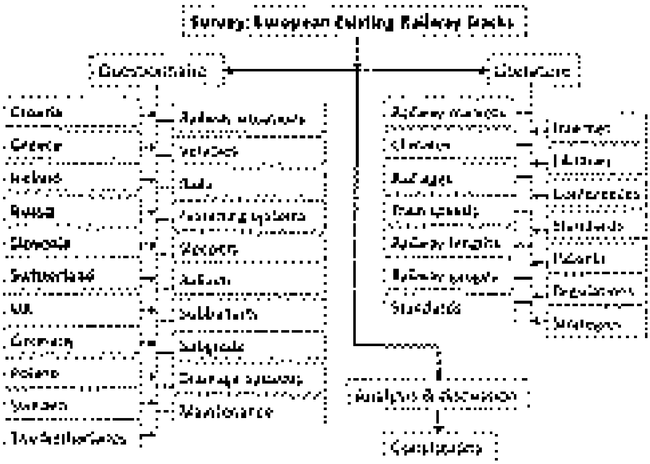

Figure 2 Map of the survey process

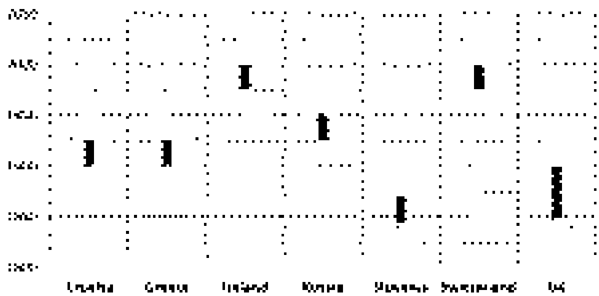

Figure 3 The age of the majority of railway infrastructure in use.

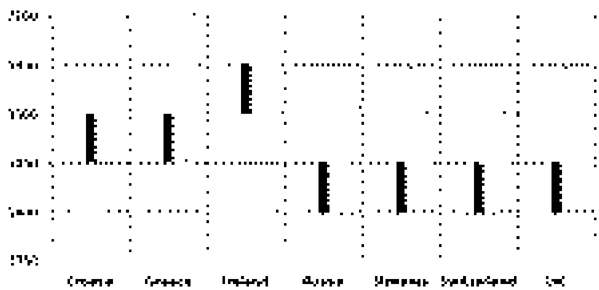

Figure 4 The oldest railway track system still in service.

When asked "what is the most frequent cause of failures on the railway track system", switches and crossings had the highest frequency of failure, followed by landslides and problems with transition zones. One railway operator specifically noted the correlation between aged or infrastructure in poor condition and failure frequency. The weakest component was generally seen to be related to the rail tracks.

The frequency of performing preventive inspection and maintenance for the railway track was very different across the RCs, from daily to weekly and half-year frequency. In Greece, surveys are conducted twice a year by a track recording car and monthly and weekly by the track agents; whilst in Slovenia, the respective rates are annualmaintenance and weekly – preventive inspection. In Switzerland, visual inspection is carried out every fifteen days and vehicle measurements are made twice a year.
The track stiffness adopted across Europe varied widley from 40 kN/mm to 200 kN/mm, See Table 1, which includes data also on the maximum axle loads by Country. In Figure 5 an overview of the current highest speed and expected future highest speed demand based on the existing railway system is presented. The data shows that in the majority of countries the current speed is between 120 km/h and 160 km/h. Most countries are planning for speeds in the range 160 km/h to 200 km/h in the future.

Table 1 Future evolutions in relation to vehicle load carrying capacity and high speed rail

	Stiffness of the existing railway tracks [kN/mm]	Max. axle load [t/axle]
Croatia	150 – 200	22,5
Greece	50-75	22,5
Ireland	-	18
Russia	100 – 150	30
Slovenia	60 – 100	30
Switzerland	100 – 200	22,5
UK	40 – 100	25

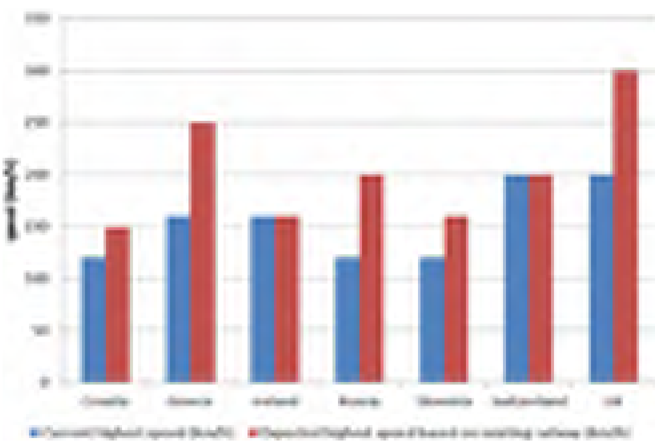

Figure 5 Highest speed of the current and expected future rail transport

The most cost-efficient solution to the railway track is generally considered to be ballasted track. The largest proportion of the RC's maintenance budget is generally expended in relation to ballast, renewal/replacement followed by rail tracks, and subgrade materials.
The average unit cost for the maintenance of railway track was quite consistent in the surveyed countries, and varied between 40 and 67 €/m. For example in Switzerland unit cost for mechanized maintenance, which include ballast distribution, tamping, compaction and stabilization is 67 €/m. Renewal of ballast, rails, sleepers without cleaning up of infrastructure is 1400 €/m, while renewal of ballast, rails, sleepers with cleaning up of infrastructure is 2500 €/m.

3.3 Results of literature survey

The literature survey was focused on the issue what the future rail transport trends are. Figure 6 shows the population per km track in these countries in a number of European countries. [4] It can be seen that surveyed countries (Croatia, Greece Ireland, Russia, Slovenia, Switzerland, UK) are covering from average to very high population densities per km track.

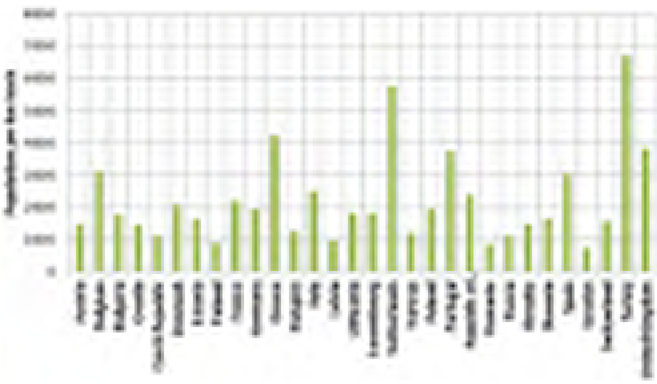

Figure 6 Population per length of railway track in selected European countries (data source: [4])

Figure 8 shows statistical data from 1980 to 2009 for railway passenger journeys in four different countries, USA, China, Japan and the UK. It can be seen that for all the four countries, railway passengers journeys and/or in length have been increasing annually. This implies that rail transport markets are increasing rapidly both in established and emerging economies.
Since 1992, traffic on the Chinese railway network has increased by more than 110 percert [5]. In 1990s, the network had a 2% average annual growth rate; while in 2000s, the growth rate increased to 8% annually for both passenger and freight. This growth actually depends upon a number of factors, for example, national economic and trade growth, market evolution and demands, railway management competence and actions, national transport strategy and funding, and the like [6].

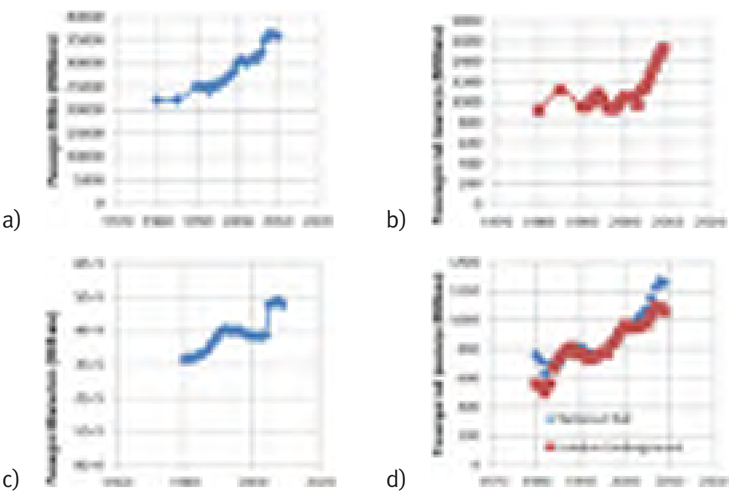

Figure 7 Statistical data from 1980 to 2009 for railway passenger journeys for four different countries: a) United States of America, b) China, c) Japan and d) the UK, [7-10]

From [4] historical data on train speeds has been collected and analysed. Before 1990, the data adopted a linear proportional increase. After 1990, however, train speeds have increased remarkably. With increasing rail market and advanced technologies, it is predicted that the train speed will continue to increase.
After analysing the related data from selected countries, it could optimistically be concluded that future railway demands will be increasing globally.

4 Conclusion

A survey on European existing railway tracks was carried out to identify challenges related to current infrastructure. There is a strong demand for rail transport and continuous research and development is being carried on to make railways faster, heavier, longer and greener. Most of the existing railway tracks in Europe have reached their design service life; the main challenge is how to enhance these existing ballasted tracks. Most countries have their own standards in railway engineering and maintenance. Some typically technical data have been obtained from seven countries which may be useful for engineering design or evaluation. The effects of climate on track services are critically significant. There are a number of projects on R&D of railways worldwide. There is a great deal of information on railways in various sources, e.g., journals and magazines, societies conferences, internet, etc. it would be helpful to obtain sufficient literature search and review before starting a new research and development project. It is found that: (a) the markets for rail transport are increasingly global; (b) with technological advances, rail transport trends are for faster, heavier and longer trains, as well as increased cost and energy effectiveness and greater reliability; (c) most of the railway tracks have reached or exceeded their design service life; (d) ballasted tracks are the most popular and economical track bed structure but there are some challenges related to how to enhance these infrastructure systems.

Acknowledgement

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References

[1] SMART RAIL FP7 project, http://smartrail.fehrl.org/

[2] D 3.1 Deliverable European Existing Railway Tracks: A Survey Report on typical problems and assessment framework for existing railways (2013), SMART RAIL, FP 7 project.

[3] Zoeteman, A. (2001), Life cycle cost analysis for managing rail infrastructure, EJTIR 1, No. 4, pp. 391-413.

[4] Wikipedia, http://en.wikipedia.org/wiki/Main_Page

[5] Scales, J. and Amos, P. (2009). An overview of China's Railway Development Program, World Bank Transport Forum, April 2009, siteresources.worldbank.org/ accessed on 18 Feb. 2013.

[6] Zheng, W. (2009). Chinese Railway Transportation: Opportunity and Challenge, http://ict.uiuc.edu/railroad/CEE/pdf/Events/TRB09/Xiang-TRB%20Paper.pdf, accessed on 18 Feb. 2013.

[7] Network Rail, http://www.networkrail.co.uk/

[8] Statistics in Rail, UK Department of Transport, http://www.dft.gov.uk/statistics/series/rail/

[9] http://www.sdrm.org/faqs/gauge/index.html, accessed on July 2012

[10] UK National Statistics, http://www.statistics.gov.uk/hub/index.html.

FINANCING OF RAILWAY CORRIDOR INFRASTRUCTURE IN TRANSIT COUNTRIES

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Abstract

The paper deals with the problems of fulfilment of the new TEN-T infrastructure guidelines in Slovenia on one hand and the financing of upgrading and construction works for the new infrastructure on the other hand. The specific of our network is that the majority of our freight traffic is in transit that means this freight has no origin or destination in Slovenia and serves mainly to the foreign economy. The fact that the origin and/or destination of most of the freight traveling on our tracks are not in Slovenia is not the problem for itself, the real problem is that the railway undertakers are not fully charged for the costs they produced to the infrastructure as well as to the environment. This is the main reason why no major railway project in Slovenia is financially proved although the traffic volume forecast are enormous.
With other words the Cost Benefits Analyses can not be positive enough because the costs are Slovenian and benefits are from the countries which transported goods through our country. New railway infrastructure can even not be proved by calculating of benefits by lowering of environmental costs with shifting the freight from the road to the rail. This is why because these environmental costs produced by the road traffic are actually also not ours, they are forced to use since also the road freight transport is on our roads mainly in transit. The solution for countries like Slovenia lies in infrastructure fees which will cover full infrastructure and environmental costs or the totally different scheme of co-financing on the European level.

Keywords: railway infrastructure, TEN-T network and guidelines, financing, transport

1 Introduction

It seems that the transport today is more and more an independent branch of economy, rather than it´s support arm. You could say that it is becoming practically an end in itself, thereby fatally affect the daily lives of people and policy, which is responsible for the development of transport, energy, environment. Transportation is constantly increasing in spite of aging and reducing the number of people in our part of the world. There are demands and the “need” for new and new infrastructure. It is needed to put an end to the myth that continued growth will never stop.
If you want to return transportation to the place it deserves in the transport chain, it is high time to start charge the transport the full costs of the construction, maintenance and management of infrastructure and management of traffic on it, as well as all costs incurred to the space and environment.

2 TEN-T infrastructure guidelines and financial consequences

The standards for the core TEN-T railway network as adopted by the Regulation (EU) No 1315/2013[1] and which should be implemented by the year 2030 are for the railway infrastructure for freight trains determined as follows:

1 full electrification of the line tracks and, as far as necessary for electric train operations, sidings;
2 freight lines of the core network as indicated in Annex I: at least 22,5 t axle load, 100 km/h line speed and the possibility of running trains with a length of 740 m;
3 full deployment of ERTMS;
4 nominal track gauge for new railway lines: 1 435 mm except in cases where the new line is an extension on a network the track gauge of which is different and detached from the main rail lines in the Union.

If we compare the aforementioned requirements for the railway infrastructure with the existing condition of the TEN-T network lines in Slovenia, we may draw the following conclusions, presented in the Table 1.
The total TEN-T network of railway lines in Slovenia is 567.1 km long, 323.2 of them are double track lines and 243.9 km are single track lines. All together 890.3 km of tracks perform the TEN-T Slovenian railway core network.
206.9 km of double track lines (64.0 %) respectively 110.2 km of single track lines (45.2 %) does not meet the requirement for the minimum speed of ≥100 km/h. Or with other words, 317.1 km of lines respectively 56.0 % of all Slovenian TEN-T network will need to be reconstructed or completely new build for achieving the desired speed of at least 100 km/h.
The first demand of TEN-T requirements, that is electrification, will be fulfilled till the year 2015, since the electrification works are in progress on the last yet not electrified line Pragersko–Hodoš on the Slovenian-Hungarian connection of the new Mediterranean railway corridor.
The Slovenian TEN-T network and plans for the future development are shown at the Figure 1 below. For the time being this development plans are not yet confirmed by the government and the Parliament, while the National Development Plan is still in the preparation phase and will be published probably in autumn this year. It is the great possibility that the development plan of the lines will stay the same but the final realisation will be postponed.

Table 1 TEN-T lines in Slovenia

Line section	Double track line [km]	Single track line [km]	V≥100km/h (2) [km]	V≥100km/h (1) [km]	V≥100km/h (2) [%]	V≥100km/h (1) [%]
Dobova–Zidani Most	50,8		33,7		66,3	
Zidani Most-Ljubljana	63,7		24,9		39,1	
Zidani Most-Maribor	91,9		38,2		41,6	
Maribor-Šentilj-s.b.		16,5		3		18,2
Pragersko-Ormož		41,2		38,4		93,2
Ormož-Hodoš		69,2		59,2		85,5
Ljubljana-Sežana-s.b.	116,8		19,5		16,7	
Divača-Koper freight st.		45,5		0		0
Ljubljana-Jesenice-s.b.		71,5		33,1		46,3
Total V≥100km/h	**323,2**	**243,9**	**116,3**	**133,7**	**36,0**	**54,8**
Total V‹100km/h	**567,1**		**206,9**	**110,2**	**64,0**	**45,2**

Figure 1 Slovenian TEN-T network

2.1 Needed investment money for realisation of TEN-T requirements

The question is how much money do we need for the fulfilment of TEN-T standards? The biggest problems are the minimum speed of 100 km/h and the useable length of station tracks of 750 m. With a rough estimation the investment costs would be between 5.5 and 7.5 billion EUR depending on the designing speed for open tracks (from 100 km/h to 160 km/h). Before beginning of construction works the National Spatial Plan (NSP) must be prepared and adopted by the government. These procedures take in average at least three years. This means that before the beginning of the year 2017 no major project can start although the NSPs for the Maribor-Šentilj and Ljubljana-Jesenice lines as well for the Pragersko station are already in progress. 14 years would be available as the realisation time. Regarding the needed finances and available time, between 390 and 530 million EUR per year should be invested into the Public Railway Infrastructure (PRI) of Slovenia to upgrade the Slovenian TEN-T network to the desired level.

3 Slovenia as a transit transport land

More than half, exactly 57.2 % of railway freight in tonnage in realised in transit transport on the Slovenian railway network in the last year, what is shown in this Section 3. The statistics is even more favourable for the transit if we take into account the NTKM. In later case the percentage is even 72.2 % (All together 4,289.8 million NTKM and in the transit together 3,095.7 million NTKM were realised).

3.1 Past freight railway traffic flows

Slovenia is a typical freight transit transport land what means that more than half of all forwarded freight is realised as a land or a port transit. Figures related to the past, by railway transported goods, are shown in the following Table 2. The figures present the total amount of all three freight Railway Undertakers (RU) operated in Slovenia.

Table 2 Transported goods by railway (in `ooo tonnes)

Year	2007	2008	2009	2010	2011	2012	2013
Dom. transport	**1,853**	**1,915**	**1,634**	**1,870**	**1,474**	**1,369**	**1,550**
- import	5,198	5,343	4,064	4,333	4,962	4,194	4,259
- export	2,260	2,089	1,861	1,994	2,018	2,063	2,391
- land transit	4,187	3,773	2,817	3,377	3,314	2,690	2,858
- port transit	5,751	5,894	5,051	6,433	6,817	7,215	8,129
Transit together	**9,938**	**9,667**	**7,868**	**9,810**	**10,131**	**9,864**	**10,987**
Int. transport	**17,396**	**17,099**	**13,857**	**16,138**	**17,111**	**16,094**	**17,638**
All together	**19,249**	**19,014**	**15,491**	**18,008**	**18,585**	**17,463**	**19,188**

From the Table 2 it is obvious that the domestic railway transport, i.e. the origin and destination of goods are in Slovenia, represents approximately 10 % of total transported goods till the year 2010 and fell even to 8 % in the years 2011 to 2013. The reason for such result lies probably in short transport distances in our country and the decline of the Slovenian industry which input raw material and the output products were dedicated for the railway transport.
It is evident that the Port of Koper is the most important O/D source for the railway freight transport on our railway network. Approximately 55 % of all loaded and unloaded freight comes to and leaves the Port by the rail. In average 30 % of the total throughput in the port has the Slovenian origin or destination and from this 30 % of Slovenian port freight about 66 % were transported by the rail in the last three years. Just for the information the data regarding the port throughput, this was in the last three years as follows: 18.0 million tonnes in the year 2013, 17.9 in the year 2012 respectively 17.0 million tonnes in the year 2011.

3.2 Future freight railway traffic flows

For the future traffic flows extensive analysis have been done in recent years. These analyses show that we can count with enormous growth of freight traffic till the year 2030. It is supposed that freight throughput in the Port of Koper will rise from about 18 million tonnes to the about 30 million tonnes. In general almost doubling of the existing railway freight transport is expected in the next 17 years in the "do minimum" scenario.
With extensive upgrading of the "Mediterranean corridor" (the speed of 100 km/h for freight trains and sufficient capacities of nodes and stations) the freight flow on the Divača–Ljubljana section could rise up to 35 million tonnes. In this case the capacity of existing double track lines would not be sufficient any more. Forecasted figures for the railway freight transport can be seen from the Table 3.

Table 3 Transported goods by railway (in `ooo tonnes)

Line sections	year 2013	"do-minimum" 2030
Koper-Divača	10,000	17,000
Divača-Ljubljana	12,000	21,000
Ljubljana-Zidani Most	8,000	21,000
Ljubljana-Jesenice	5,000	9,000

3.3 Infrastructure users fees

In the Table 4 the amount of paid infrastructure users' fee in the last three years are shown separated by users. At the moment four RU have Slovenian Safety Certificate and are active in the transport industry. Three of them are freight RU.

The coverage of the PRI costs for the maintenance and traffic control and command (TC&C) leis only between 8.5-9.5 %. The costs for renewals, upgrading and new construction of the PRI are even not considered in the stated percentages. In the Republic of Slovenia the infrastructure fee is obligated only for the freight trains and special passengers trains but not for the trains running under public service obligations contract.

Table 4 Infrastructure users' fees (in 000 EUR)

Railway Undertaker	2011	2012	2013
Rail Cargo Austria AG	447.748,13 €	628.657,26 €	685.983,61 €
Slovenian railways, Freight Transport, ltd.	8.515.154,64 €	7.604.156,50 €	8.117.812,12 €
Slovenian railways, Passengers Transport, ltd.	53.711,20 €	43.841,71 €	51.055,76 €
Adria Transport, ltd.	112.841,23 €	169.604,07 €	273.407,49 €
Without V.A.T.	**9.129.455,20 €**	**8.446.259,54 €**	**9.128.258,97 €**
With V.A.T.	**7.607.879,33 €**	**7.038.549,61 €**	**7.540.713,31 €**

From the above percentage it is obvious that we are far away from the full coverage of the costs which transport causes to the infrastructure and to the environment. It is also true that also road traffic is not paying the infrastructure costs caused to the Slovenian environment.
A simple comparison between EUR/NTKM paid by the freight railway transport and the road tax paid for yearly registration of passengers cars in Slovenia shows firstly how small railway fee actually is (3,781millionNTKM realized in the year 2013 and 9.128 million EUR infrastructure fee means 0.0024 EUR/NTKM) and secondly that even road tax for passengers cars are almost double as high as the infrastructure fee (road tax for average passengers car of 1350 cm^3 and weight of 1 tonne is 62 EUR; with in average 15,000 km/year it means 0.0041 EUR/NTKM).

Table 5 Maintenance and TC&C costs of PRI (in 000 EUR with V. A.T.)

Year	2007	2008	2009	2010	2011	2012	2013
Maint. of PRI	69.370	73.801	67.717	60.928	71.589	68.039	63.787
TC&C	41.722	36.864	37.849	35.916	34.500	33.000	33.000

4 New EU infrastructure financing policy

4.1 What can we expect in the new financial perspective from 2014 till 2020?

First of all the old financial instrument TEN-T is replaced with the new scheme called "Connecting Europe Facility" (CEF)[2], which will assure the financial means for the actual perspective in the period from 2014 till 2020.
In the most radical overhaul of EU infrastructure policy since its inception in the 1980s, the Commission has published new maps showing the nine major corridors which will act as a backbone for transportation in Europe's single market and revolutionise East–West connections. For realising of this ambition, EU financing for transport infrastructure will triple the finance to the 26 billion EUR from which about 11.3 billion EUR has been taken away from the Cohesion found and will be dedicated only for the Cohesion Countries upon the public tenders.
This new infrastructure policy will put in place a powerful European transport network across 28 Member States to promote growth and competitiveness. It will connect East with West and replace today's transport patchwork with a real European network.

The new policy establishes, for the first time, a core transport network built on nine major corridors: 2 North–South corridors, 3 East–West corridors; and 4 diagonal corridors. The core network will transform East–West connections, remove bottlenecks, upgrade infrastructure and streamline cross-border transport operations for passengers and businesses throughout the EU. It will improve connections between different modes of transport and contribute to the EU's climate change objectives. The core network is to be completed by 2030.
This EU funding will be tightly focused on the core transport network where there is most EU added value. To prioritise the East–West connections, almost half the total EC transport infrastructure funding (€11.3 billion from the "Connecting Europe Facility", or CEF) will be ring-fenced only for cohesion countries, [3].

5 EU co-financing of the PRI in the Republic of Slovenia

5.1 Last financial perspective 2007-2013

In the last financial perspective Slovenia has got about 450 million EUR from the Cohesion Found and some project design documentations co-financed from the TEN-T source. The total amounts of money spent for the investments into the PRI, from the year 2009 to the year 2012, are as follows (in 000): in the year 2009 – 100,115 EUR, in the year 2010 – 131,036 EUR, in the year 2011 – 105,728 EUR, in the year 2012 – 71,559 EUR, in the year 2013 – 133,462 EUR and for the year 2014 – 336,340 EUR is planned.
For the year 2015 approximately the average sum of last years will be needed to complete all foreseen projects in time. All above presented figures are the sum of European Cohesion founds and our own means.

5.2 New financial perspective 2014-2020 (railway sector)

Slovenia will probably get round 360 million EUR from the Cohesion envelope. From the special IPE quota for Cohesion countries Slovenia is entitled to an amount of 159 million EUR which must be tied up till the end of 2016.
As it was said before for the IPE instrument public tenders will be launched. How much Slovenia will be able to get from the IPE will depend on the quality of the proposed projects sent to the tenders launched by the EC.
At the moment no official figures regarding Slovenian forecast for the amount of money from the IPE instrument are available. The unofficial information said that for the next legislative period from 2014–2020 Slovenia plans to get from all European founds together (Cohesion found, ERDF, IPE) and for the whole Traffic infrastructure in total about 1,065 million EUR. Our own participation should be at least in the same value.
From the above money about 765 million EUR should be the investment money for the PRI. Together with our participation it presents about 1,530 million EURs investment money for the PRI.
From the above figure it is possible to calculate the average yearly capability of investment many in Slovenia which lies between 200 and 250 million of EURs, what is far away from the needed investment money calculated in the Section 2.1.

6 Conclusions

It is obvious that yearly paid infrastructure fees for using of the PRI in Slovenia are not high especially in the comparison with the costs of the maintenance of the PRI and of the TC&C on it, which are shown in the Table 5. The result of such low percentage of operator´s contribution to the basic cost on the PRI are probably due to not paying of environmental costs at all and in the too low infrastructure fees in the transport in general.

It is simply not possible with such infrastructure users' fees, for the countries like Slovenia, to finance the appropriate renewal, upgrading and adaptation of the capacity needs to the market demands anymore.
The transport charges and taxes must be restructured in the direction of wider application of the 'polluter-pays' and 'user-pays' principle (at least for the freight transport and to cover the real costs of the PRI and the TC&C on it). They should underpin the transport's role in promoting European competitiveness and cohesion objectives, while the overall burden for the sector should reflect the total costs of transport including infrastructure and external costs, [4].
Putting an end to the permanent changing of the European priority corridors, which are mostly determined on the political level as a compromise and not necessarily correspond to the real facts and needs. The infrastructure corridors should be developed on the European level upon the real wider economic needs and not on the basis of single member state priorities.
Financing of the projects should be released of the unnecessary administration and numerous controls, which take time and cost money. Project management should be involved instead of administrating. Appropriate amount of investment money from the EU funds must be dedicated to the single project upon the Feasibility Studies directly without the tendering. Totally different scheme of co-financing on the European level are needed.
Decisive measures on the EU level will be needed to restrict the long haul road freight transport, because the soft measures proven didn't bring the modal shift from road to rail. Although the rail is amongst the most efficient and climate-friendly type of transport it currently carries about only 10% of European cargo and 6% of passengers each year, [5].

References

[1] Regulation (EU) No 1315/2013 of the European Parliament and of the Council of 11 December 2013 on Union guidelines for the development of the trans-European transport network and repealing Decision No 661/2010/EU.

[2] Regulation (EU) No 1316/2013 of the European Parliament and of the Council of 11 December 2013 establishing the Connecting Europe Facility, amending Regulation (EU) No 913/2010 and repealing Regulations (EC) No 680/2007 and (EC) No 67/2010.

[3] Transport: New EU infrastructure policy http://europa.eu/rapid/press-release_IP-13-948_en.htm?locale=en; 17.10.2013.

[4] EC WHITE PAPER, Roadmap to a Single European Transport Area – Towards a competitive and resource efficient transport system; Brussels, 28.3.2011.

[5] EU triples funding for rail innovation http://ec.europa.eu/transport/index_en.htm; 16.12.2013.

THE STRATEGY OF INTRODUCING ECTS SAFETY SYSTEM ON RAILWAY CORRIDOR Vc IN BOSNIA AND HERZEGOVINA

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Abstract

ETCS (European Train Control System) is a unique European standard for controlling and managing a railway traffic. It was a result in a form of need for mutual railway abovestandard in European countries which has a goal to adjust the existing systems of railway managing and to create a unique European system of insurance which has a result to creat a unique European railway network. Beside the mentioned reasons for introducing the ETCS system other reasons are the increasing diversion of cargo flows from road to rail. To implement the ETCS system on BH railways at Corridor Vc it is necessary to analyse present state of railway infrastructure and to adjust and renew infrastructure so it would be prepared for installation of ETCS system. In this paper a possibility of introducing of this security on BH railway at Corridor Vc was presented. Implanting of ETCS security system was presented throughout four project options. Project options allude implementation of ETCS separatly for each level of that system. First project option refers to implementation and modernization of current degree of insurance so that Corridor Vc could be lead to ETCS Level 0. 2nd, 3rd and 4th option is concerning on ability of introducing higher levels of ETCS which are ETCS Level 1, ETCS Level 2 and ETCS Level 3. Whith evaluation of project options analysis it was concluded that each design option is reasonable, cost-effective and profitable.

Keywords: ETCS, BH railways, Corridor Vc, ETCS Level

1 Introduction

The best way to carry out modernization and standardization of infrastructure, is the introduction of ETCS security system. Strategy of introducing other ETCS on B&H's railroads on Corridor Vc in this paper is presented with four project options starting with ETCS level 0 up till level 3. Project options are divided by degree or level of ETCS insurance. The first project option involves the reconstruction of infrastructure by bringing it to the national level of security that meets ETCS Level 0. Second project option considers introducing of ETCS Level 1, while the third and fourth project options consider the possibility of upgrading the existing ETCS Level 1 to higher levels the ETCS Level 2 and Level 3.

1.1 Status line at Corridor Vc in Bosnia and Herzegovinia

Corridor Vc is setted from Budapest up tilland Ploče. At mostly it pasess through Bosnia and Herzegovina (405 + 741 km). Corridor Vc enters Bosnia and Herzegovina in Bosanski Šamac where it is connected to the track line Bosanski Šamac – Sarajevo. From Sarajevo it follows the line Sarajevo – Čapljina – state border.
Railroad line Bosanski Šamac – Sarajevo was built from April 1th until November 15th 1947. The total length of this line is 235 + 351 km. On this line there are 63 official locations, 20 of

them are stations, two of them are main, Doboj (ŽRS) and Sarajevo (ŽFBH) On this line there also are 33 train stops, 8 passing point, a junction and a state border.

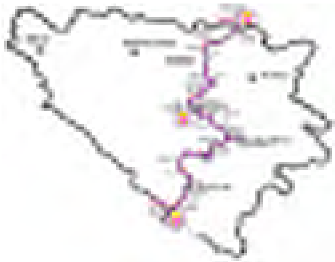

Figure 1 Route of Coridorr Vc through Bosnia and Herzegovina [1]

Line Sarajevo-Čapljina, to be more accurate line Sarajevo – Čapljina – state border – Ploče began in construction in 1958 and its construction was completed in 1966. Total length of this railway line is 170 + 390 km. On this line there are 41 official locations, of which are 16 stations of which one is the main station named station Sarajevo. The railway also has 16 train stops, 7 passing point, one junction and one national border, [1].

1.2 ETCS development

As part of UIC's projects for infrastructure, largest and most important project is – ERTMS (European Rail Train Management System). Key projects, from a technical point of view, that are incorporated into ERTMS are: European Train Control System – ETCS and Global System for Mobile Communication – Railway – GSM-R. ETCS is formed with four levels, (Level 0, Level 1, Level 2 and Level 3). Goals of ETCS system, according to [2]:
· centralize management and make it more intelligent;
· reduce the costs of operation and maintenance of stationary systems;
· increase the interoperability of heterogeneous railway systems;
· increase the capacity and speed.

Functions of ETCS system:
· monitoring of (local) maximum speed;
· monitoring the correct route of trains movement;
· monitoring the direction of movement;
· monitoring the implementation of special regulations.

2 ETCS Levels

2.1 ETCS Level 0

When the vehicle that supports ETCS is being used on a line that does not support ETCS, that is Level 0 of ETCS.

2.2 ETCS Level 1

ETCS Level 1 system is a signalling system that allows the integration of ETCS equipment on existing national signalling security system that remains in operation. At this level on rail tracks Eurobalise are being installed, those transmitters take on signals from the existing track signalling via signal adapters and telegram coders (LEU – Lineside Electronics Unit) and pass them as data to train as data that gives authorization to move and contains information on route and fixing points. Because of a point distribution of these transmitters, it is necessary that train passes over this transmitter first in order to obtain information about the next section. With cable connection of these transmitters, known as EuroLoop, constant communication with the vehicle is being held. Euroloop system implies the installation of power lines which transmit signals along the rails so that it achieves continuous coverage of the driving path, which increases safety and reliability of traffic.

Figure 2 Schematic diagram of ETCS – Level 1[1]

2.3 ETCS Level 2

ETCS – Level 2 is a system oriented to the digital radio transmission signal and train protection system. ETCS – Level 2 is a practical upgrade of ETCS – Level 1 with all the Level 1 elements but with addition of GSM-R transmitter on the locomotive, the GSM-R receiver and Radio Block Centre (RBC). Eurobalise transmitters are used at this level only as passive positional elements or so-called Electronic milestones. Between the two positional elements train defines its position by sensors. This level system is based on radio. It uses the GSM-R system, in order to continuously transferred speed limit information on the route to the vehicle. At this level Eurobalise are used as passive positioning markers.

Figure 3 Schematic diagram of ETCS – Level 2 [1]

2.4 ETCS Level 3

Level 3 of ETCS system goes considerably beyond the simple signal-safety management and navigation, because it is fully implemented in the radio control system distances between trains. Fixed rail signalling devices are no longer needed. Trains define its positions the same as in Level 2 through sensors. As a participant in traffic here the train is given with the autonomy of making decisions, therefore, attention must be given to maintaining a high level of reliability. Level 3 is still under development and in Europe there are currently several experimental and pilot projects in several railroad stocks. This level supports full radio signalization, [3].

Figure 4 Schematic diagram of ETCS – Level 3 [1]

3 Strategy of ETCS implementation at Corridor Vc in B&H

3.1 Strategy of ETCS Level 0 implementation

This level of rail security will lead to increased safety and increased line capacity in facilitation of rail traffic where this level was the basis for the introduction of ETCS Level 1 ETCS as a basic level of ETCS security. The costs of reconstruction and installation of complete SS and TT security system at complete section of the Corridor Vc is estimated by former cost experience at implementation of such security levels of insurance in relation to the length and characteristics of the rail. December 30th 2009 ŽFBH signed an agreement with German company Thales Rail Signalling Solutions GmbH on support and installation of SS and TT equipment for line Konjic – Čapljina in value of €11,742,602,72 million. This section of rail is single-railed with length of 96 + 466 km and it contains 22 official locations of which is 10 stations, 4 crossing and 8 train stops.

Table 1 Economic analysis and project appraisal of ETCS implementation on B&H's rails on Corridor Vc – Option 1 – ETCS Level 0 [1]

Year	Eeconomic costs of investment (€)	Savings in maintenance costs (€)	Savings in staff costs (€)	Cost savings in traveling time for passengers and goods (€)	Savings in the costs of train operation (€)	Benefits of introduction of ETCS Level 0 (€)	Benefits and costs flow (€)
1	-26.660.000						-26.660.000
2	-26.660.000						-26.660.000
3	-26.660.000						-26.660.000
4		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
5		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
6		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
7		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
8		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
9		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
10		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
11		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
12		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
13		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
14		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
15		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
16		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
17		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
18		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
19		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
20		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
21		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
22		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
23		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
24		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
25		-550.000	624.000	11.160.000	-1.499.712	2.571.072	9.734.288
Residual value		7.998.000					7.998.000
EIRR							9,73%
NPV (5%)							40.333.206

According to this ratio deploy cost of insurance implementation and length of rail on which the implementation is being done, it can be concluded that the total cost for implementation of the SS and TT insurance on complete length of the rail at Corridor Vc (405 + 741 km) through Bosnia and Herzegovina is on average of € 50 – 60 million. Locomotive expenses or costs of installation the ETCS equipment in locomotives are around €100.000 when it comes to new locomotives. Railways in Bosnia and Herzegovina do not dispose with nither one new locomotive, so the investment costs of ETCS equipment installation in existing engines are significantly higher on average of €2.00.000 – 250.000 depending on the state of the locomotive. Considering the fact that these are relatively old locomotives costs of ETCS equipment installation per locomotive is an average of around € 200.000. From this facts it can be concluded that total cost of ETCS equipment installation for the entire locomotive fleet of 80 locomotives at Corridor Vc is around € 15-20 million. In this option, as this will be the case in other options, costs of maintainance will be analyzed for 25 years from implementation of ETCS system Level 0. So it can be concluded that the overall costs for implementation of ETCS Level 0 insurance at Corridor Vc on Bosnia and Herzegovina's rails on average is around €70 to 80 million.

3.2 Strategy of ETCS Level 1 implementation

Costs of ETCS Level 1 implementation are divided into costs that are related on equipment installation and costs of infrastructure and equipment maintenance. The costs of preparing tracks for installation of ETCS system Level 1, according to the the experience of countries in which this level is installed (Austria), are around € 2.000 – 3.000 per mile of track. B&H's railways on Corridor Vc have a length of 405+741 km therefore the costs of preparing the route for the introduction of ETCS Level 1 is to about € 1.000.000. As the cost of installation and deployment of ETCS rail security Level 1, are ranging up till € 30.000 – 300. 000 per km of railways, it is important to note that these costs far exceed the upper limit of € 300 000 when it comes to installation of the equipment in the station area, while costs on the open line are considerably less than € 30 000 per km of track. According to empirical examples (Austria), it can be assumed that costs of ETCS Level 1 equipment installation in large stations is a around € 3-4 million. In smaller stations the empirical examples come to the figure of about € 1 to 1.5 million per station, while the cost of ETCS equipment installation on train stops, junctions, level crossings, passing points, and industrial gauges are around € 200.000 – 400.000 depending on the characteristics of train stops and level crossings. While the cost of ETCS equipment installation on the open railroad on average does not exceed € 50. 000 – 80.000 per track mile. Out of 36 stations at the Corridor Vc there is total of 6 large stations: Bosanski Šamac, Doboj, Zenica, Sarajevo, Mostar and Čapljina station. This means that total cost of ETCS Level 1 equipment installation at these stations is equal of € 20 million. Cost of equipment installation at the remaining 30 stations are around € 30 million. Total cost of ETCS Level 1 equipment installation in the stations at the Corridor Vc is amount of around € 50 million. Cost of ETCS Level 1 equipment installation at other 68 official locations, industrial railtracks and a few dozens of level crossings are approximately around € 25 million. Total costs of ETCS equipment installation at the official positions is roughly around € 75 million. Finally, it can be concluded that the costs of ETCS Level 1 implementation on the BH railroads at Corridor Vc is a total of around € 100 to 120 million.

Table 2 Economic analysis and evaluation project of implementation of ETCS safety on B&H's rails at Coridorr Vc – Option 2-ETCS Level 1 [1]

Year	Eeconomic costs of investment (€)	Savings in maintenance costs (€)	Savings in staff costs (€)	Cost savings in traveling time for passengers and goods (€)	Savings in the costs of train operation (€)	Benefits of introduction of ETCS Level 0 (€)	Benefits and costs flow (€)
1	- 40.000.000						- 40.000.000
2	- 40.000.000						- 40.000.000
3	- 40.000.000						- 40.000.000
4		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
5		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
6		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
7		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
8		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
9		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
10		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
11		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
12		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
13		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
14		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
15		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
16		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
17		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
18		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
19		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
20		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
21		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
22		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
23		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
24		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
25		- 1.800.000	1.560.000	24.180.000	-9.185.736	4.138.566	14.754.264
Residual value		12.000.000					12.000.000
EIRR							9,85%
NPV (5%)							62.211.226

3.3 Strategy of ETCS Level 2 implementation

Also implementation of ETCS Level 2 costs are presumably calculated over the empirical data. For example, the costs of implementation of ETCS Level 2 system, which will be operated in Hungary by the German Siemens at line Budapest – Kelenföld – Székesfehérvár, in a distance of 65 km, costs € 53 million. According to previous experiences of countries that have implemented ETCS Level 2, on the roadlines at the Corridor Vc in Bosnia and Herzegovina at least 5 radio block centers will be need, which are placed every 50-100 km depending on the characteristics of lines and terrain on the route of the railroad. Construction and installation of radio block equipment for one radio block center will cost tens € millions. Building up and setting up the GSM-R antennas costing tens of € thousands per transmitter including signal boosters and accessories. From what is stated it can be concluded that the total cost of ETCS Level 2 implementation at the B&H's railraods at Corridor Vc in a distance of 405 + 741 km amounts to roughly € 300 million. Locomotive expenses are related to the installation of

GSM-R receiver (antenna) in the locomotives. In our case, we consider the total fleet of 80 locomotives and from the empirical data, total cost of GSM-R equipment installation to the locomotives is around € 2 million. Very expensive and high quality ETCS Level 2 equipment maintenance costs are minimized as far as possible. Thus, the annual maintenance costs should not exceed the figure of about € 2 million. Finally, ETCS Level 2 security on B&H's rails at the Corridor Vc implementation costs are total of around € 300 – 350 million.

Table 3 Economic analysis and evaluation project of implementation of ETCS safety on B&H's rails at Coridorr Vc-Option 3- ETCS Level 2 [1]

Year	Eeconomic costs of investment (€)	Savings in maintenance costs (€)	Savings in staff costs (€)	Cost savings in traveling time for passengers and goods (€)	Savings in the costs of train operation (€)	Benefits of introduction of ETCS Level 0 (€)	Benefits and costs flow (€)
1	- 110.000.000						- 110.000.000
2	- 110.000.000						- 110.000.000
3	- 110.000.000						- 110.000.000
4		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
5		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
6		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
7		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
8		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
9		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
10		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
11		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
12		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
13		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
14		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
15		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
16		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
17		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
18		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
19		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
20		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
21		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
22		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
23		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
24		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
25		- 2.450.000	3.120.000	43.524.000	- 8.248.416	9.598.896	35.945.584
Residual value		33.000.000					33.000.000
EIRR							8,46%
NPV (5%)							118.449.989

3.4 Strategy of ETCS Level 3 implementation

Level 3 is still under development and in Europe there are several experimental and pilot projects currently in several railroad stocks. ETCS Level 3 solutions for reliable train control are very complexed and not suitable for transfer and implementation to the older models of railway vehicles. As this level is still in an experimental stage, it will at some European rails be implemented in future 20 years, which tells us that the railroads in Bosnia and Herzegovina, will not take hold in the near future.

Conclusions

Option 1, which includes reconstruction and an upgrade to the existing state of the railway security with fully restored SS and TT devices that would lead rail to the ETCS Level 0and indicates the real socio-economic benefits that are result of ETCS Level 0 implementation. This project option demonstrates an acceptable net present value and economic internal rate of return of 9.73% (EIRR = 9.73%). Also the cost-benefit ratio is larger than 1 which indicates that the project is cost effective and profitable, and that its implementation needs to begin.
The second project option involves an upgrade from the existing national railway security system and ETCS Level 1 implementation. This is financially much larger project compariong to the project of ETCS Level 0 implementation but in the end there are much greater benefits from this project or from this ETCS level. This option also displays projects reasonable net present value and also an acceptable value of economic internal rate of return (EIRR = 9.85%).
The third project option requires substantially greater financial investment because ETCS Level 2 involves the usage of GSM-R system and with this level makes the European Rail Traffic Management System (ERTMS). However, by analyzing the relation between costs and benefits and the possibility of introduction of ETCS on the BH's railways at Corridor Vc is reasonable and shows a positive net present value and internal rate of return equal to 8.46% (EIRR = 8.46%).
The fourth design option involves the introduction of ETCS system Level 3, which represents the highest possible level of ETCS security. Implementation of this system in the EU is not expected before 2025. Finally, it can be concluded that the project of ETCS implementation on BH's railways at Corridor Vc is fully justified, effective and profitable.

References

[1] Marković, I.: Analysis of the possibility of introducing ETCS insurance on BH Railways on Corridor Vc, Master's thesis, Faculty of Traffic and Communications Sarajevo, 2011.

[2] Jenkins, P.: Railway Communication System GSM-R/FTN, Network Rail GB 2006.

[3] http://www.ertms.com

2 ROAD INFRASTRUCTURE PROJECTS CONSTRUCTION, MAINTENANCE AND MANAGEMENT

TOWARDS MAXIMIZATION OF THE ADDED VALUE OF STRATEGIC INFRASTRUCTURE PROJECTS IN SOUTH EAST EUROPE THROUGH IMPROVEMENTS AT BORDER CROSSING POINTS

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Abstract

South East Europe (SEE) is a particular region of Europe regarding transport infrastructure development and international transport operations, due to the fact that the region includes an interrupting zone of the Trans-European Transport Networks (TEN-T), namely the Western Balkans (WB) countries. Concerning strategic transport infrastructures development, substantial investments have been made during the last decades or are underway. In the current period of economic recession, the TEN-T programme and the programme for the development of the SEE Comprehensive Transport Network still promote large infrastructure projects, but seek to ensure maximum return of investments. Major obstacle for the maximisation of the benefits and return of investments, but also for increased accessibility and mobility, is the existence of borders along the main international routes crossing SEE. In mid-2013 Croatia accessed European Union and the same is foreseen for the rest of the WB countries, but with it is unknown when this will be realised. Therefore, borders will remain for long time ahead and continue to impede international transport. In the framework of "Accessibility improved at border CROSsings for the integration of SEE" project (ACROSSEE) a large survey campaign has been performed, in order to register the current situation at several Border Crossing Points (BCPs) of the region. Specifically, questionnaire-based and field surveys have been performed at 30 Road and 29 Rail BCPs, which comprised observations of border procedures, registration of waiting and procedural times, interviews of BCPs authorities and truck drivers, etc. This paper presents the surveys' methodology and preliminary results, targeting at defining short term measures and recommendations for the facilitation of trade and transport in SEE.

Keywords: transport infrastructure, border crossings, international transport and trade facilitation, South East Europe, ACROSSEE

1 Introduction

SEE region is still a discontinuity zone of the TEN-T. The authors in the recent past [1] discussed about the planning experience and the perspectives for strategic transport infrastructure development in the region in the context of European Transport Policy, based inter alia on the SEE Transport Axis Cooperation (SEETAC) project [2]. SEETAC (a project also co-funded by the SEE Programme), elaborated while the TEN-T new regulation was still under preparation, concluded to priority transport infrastructure projects, but moreover to the ascertainment that large scale infrastructure projects implementation in a period of scarcity of funds should be very carefully planned, considering that capacity limitations are rare in the region and that it is very important to tackle the problems of interconnectivity of the SEE network with the TEN-T

and the problems at border crossings. In other words, why spend millions in large infrastructures that have long preparation period under the current circumstances just to save minutes in travel time and why not reduce times at borders from hours just by half in the short term?
While for the first time the TEN-T guidelines [3] include indicative maps of the TEN-T extensions in SEE and indicating the important interconnecting points, the ACROSSEE project, co-funded by the SEE Transnational Cooperation Programme [4], had set an ambitious goal to survey and identify the problems for international transport, with special focus on border crossings.
The project's technical activities are divided in three main pillars: a) the establishment of an institutional platform and enhanced administrative cooperation for transport development and facilitation of movement of passengers and goods in the region, b) the development and use of a transport model to assess the existing and future bottlenecks and the needs for short term improvements and c) the analysis of border crossings in order to identify problems and measures to ease up international traffic.
This paper presents the methodology and preliminary findings of the cross border surveys and analysis in the framework of ACROSSEE.

2 Methodology for the surveys

The surveys in the framework of ACROSSEE had been planned to serve both the needs of the analysis of border crossings and the transport model development. Especially for the latter, it is quite common that the border authorities and national agencies provide traffic data that most of the times differ. Therefore, it was decided to perform two-day traffic counts at Road BCPs and five-day traffic counts at Rail BCPs, to capture the rail traffic irregularities during a working week.
These traffic surveys have been only part of an extensive field survey, which was finally conducted by the project partners in each country at selected BCPs along the main international transport corridors in the region, in the periods May – June and September – October 2013, i.e. excluding the summer season. The surveys' part regarding BCPs operation comprised autopsies and questionnaire-based surveys addressed to the authorities present at each BCP and to truck drivers. In parallel, questionnaire-based survey addressed to the National Customs Agencies and the ministries responsible for borders and Customs of the involved countries was conducted (this way they were informed also about the BCPs surveys and on-site visits). Other relevant questionnaire-based surveys also performed concerned the different stakeholders, transport operators and enterprises active in SEE.

2.1 Field of survey

A major issue to confront during the planning of the surveys was the different statuses of the various SEE countries (EU member state, Schengen Treaty country, non EU member state). As a result different approaches were followed in each case: In the case of two bordering Schengen countries only traffic flows data could be performed, since stops are not obligatory and no controls are performed at those border points. In all other cases, of two neighbouring non-Schengen countries, of an EU country or a Schengen country with a non-EU country the entire set of surveys had been planned to be implemented.
As a result, the surveys had been planned to be performed at 35 Road BCPs and 30 Rail BCPs, as listed in Table 1 and presented in Figure 1. However, the BCPs actually surveyed are less, mainly due to the fact that in specific cases the surveys couldn't be performed due to unforeseen circumstances, misspecification of the BCPs names or actual operation (e.g. Vidin – Calafat Danube Bridge II was not yet operational, another BCP was dedicated only to passenger vehicles, etc.).

Table 1 BCPs selected to be surveyed in the framework of ACROSSEE

No	Country/ Bordering Country	Rail BCPs	Road BCPs
1	SI/ HR	Dobova/ Savski Marof	Obrezje/ Bregana
2	HR/ RS	Tovarnik/ Sid	Lipovac/ Batrovci
3	RS/ fYRoM	Ristovac	Preševo
4	AL/ ME	Bajza	Hani I Hotit
5	EL/ FYROM	Idomeni	Evzonoi
6	EL/ BG	Promachonas/ Kulata	Promachonas/ Kulata
7	EL/ AL	–	Krystallopigi/ Kapshticë
8	EL/ AL	–	Kakavia/ Kakavijë
9	RS/ BG	Dimtrovgrad/ Dragoman	Gradina/ Kalotina
10	BG/ RO	Vidin/ Calafat (not operational)	Vidin/ Calafat
11	RO/ BG	Giurgu/ Ruse	Giurgu/ Ruse
12	RO/ MD	Holboca	Vama Albita
13	HU/ UA	Záhony	Záhony
14	RO/ HU	Curtici/ Lőkösháza	Oradea-Bors/ Ártánd
15	HR/ HU	Koprivnica/ Gyékényes	Gorican/ Letenye
16	HR/ BA	Slavonski Šamac	Županja
17	RS/ HU	Subotica/ Kelebia	Horgoš/ Roszke
18	RS/ BA	Brasina	Trbusnica
19	RS/ ME	Koprovat	Vrbnica
20	RS/ RO	Vatin/ Stamora Moravița	Vatin/ Moravița
21	BG/ TR	Svilengrad	Kapetan Andreevo
22	UA/ RO	Dorneşti	Siret
23	AL/XK	–	Morine

Figure 1 BCPs surveyed in the framework of ACROSSEE

2.2 Tools used

The questionnaires were developed on the basis of those used in relevant surveys performed in the past by the same team at the Road and Rail BCPs along the Pan – European Corridor X [5], and thus their effectiveness had been successfully tested. These questionnaires were further improved, in order to cover the scope of the surveys; for example they included a part referring to the "mirror" BCPs, to cover the cases that they were not surveyed because not all SEE countries are represented in ACROSSEE project.

2.2.1 BCPs' Authorities questionnaires

The questionnaires addressed to the authorities of the BCPs were obviously different for the Road and Rail BCPs, following however a similar structure, in parts with thematically more or less common requested information:

The first part of the questionnaires concerned the general description of the station's infrastructure (available lanes/ tracks and their distribution, buildings, power and water supply, lighting), facilities (banks, restaurants, coffee-shops, duty free shops, etc.) and equipment (X-Rays machines, weighbridges, telephone and internet connections, tracing means, service lines and storage areas); including an assessment of their condition.

The second part referred to the operation of the station. Specifically, it included details about the agencies present (Customs, Border Police, Veterinary and Phyto-sanitary services), about their working hours and their sufficiency (in terms of number of staff, level of knowledge of computer and English).

Moreover, this second part incorporated questions in tabular form, concerning the minimum, average and maximum waiting and procedural times per type of vehicle/ train and the times. It also included questions regarding the foreseen procedures and their sequence, and details such as if some of the procedures are performed simultaneously or at separate areas. Furthermore, information concerning the cooperation and communication of the BCP authorities with those of the neighbouring BCP were also enquired.

The third part contained statistics requirements concerning traffic flows of the previous years (on annual basis), traffic characteristics (e.g. peak periods during day/week/month/year), percentage of transits, percentage of empty trucks, percentage of traffic requiring phyto-sanitary or veterinary controls, etc.). Finally, a request for estimation of the BCP's capacity usage by the authorities was included.

The fourth and last part of the questionnaires was specifically dedicated to the level of cooperation with the adjacent BCPs and the identification of any problems occurred at the station from the performance of the neighbouring BCP. Additionally, the authorities were asked to state the main problems of the BCP and about future development plans.

2.2.2 National Customs Agencies questionnaires

The questionnaire addressed to the National Custom Agencies has been developed in order to obtain information on the legislation under which the BCPs perform, the policies adopted and applied (including implementation of Integrated Border Management strategies) and finally the best practices application on issues concerning the BCPs spatial and internal organization and operation.

2.2.3 Truck drivers questionnaire

Finally, the questionnaire for truck drivers included questions about the type and weight of transported commodities, the origin and destination points and the route followed for the specific trip but also for other trips of the same driver, the waiting time and the time needed for the procedures to be performed and the problems perceived by the driver at the specific BCP as well as other BCPs along the route.

2.3 Application of the tools

It has been planned from the beginning that the surveys at BCPs should be combined with the traffic counts and therefore, in order that the BCPs authorities would have prepare the statistics data requested, the questionnaires addressed to the BCPs and the National Customs Agencies had been sent out initially and then the field surveys (traffic counts, autopsies, interviews) followed.
All data collected by the various partners have been submitted in appropriate templates and an integrated database for all BCPs has been developed for the analysis. The analysis, especially of the data concerning waiting times, procedural times and the main problems of the BCPs, included comparison of the data collected by the BCPs authorities with the respective data collected from the National Custom Agencies and other sources (e.g. enterprises, transport operators and associations) and – especially for the cases of Road BCPs – from the drivers of commercial vehicles crossing the BCPs. Most important results of the on-going analysis are presented in the following paragraphs.

3 Data analysis

The results of the analysis are presented in the series of figures below and refer to the pairs of Road and Rail BCPs as presented in Table 1 and Figure 1. Figure 2 illustrates the evaluation of the level of equipment and installations for 29 Rail BCPs and 29 Road BCPs (where relevant data have been obtained). The majority of the Road BCPs are experiencing lack of equipment of phyto-sanitary controls. Moreover, the existence of the necessary tracing means at Road BCPs increase the time for the controls. The equipment and installations of Rail BCPs overall are in good level, except the equipment of phyto-sanitary controls, which do not exist.

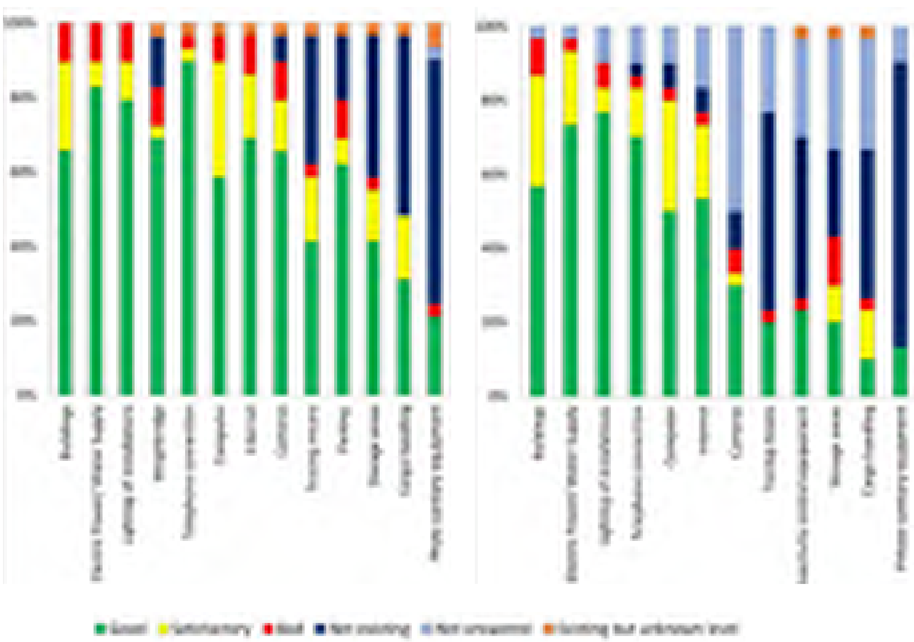

Figure 2 Aggregation of Road (right) and Rail (left) BCPs self-evaluation on the level of existing infrastructure and equipment

Figure 3 presents the Road and Rail BCPs evaluation of capacity usage (%) on daily basis, as assessed and estimated by the BCPs authorities. Overall, many Rail BCPs have higher unexploited capacity due to the low traffic and many Road BCPs cannot serve more traffic (even it is low) due to the existing problems, mainly regarding understaffing and lack of equipment.

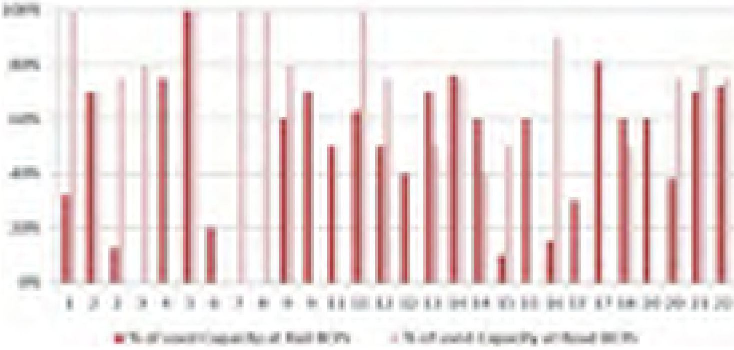

Figure 3 Single BCPs self-evaluation of daily capacity usage (%)

In Figure 4 are presented the total time for crossing a pair of BCPs as the average of the times required to do this in both directions. Where data refer to only one BCP (and not a pair) this is indicated by an asterisk (*), while if no data exist for a BCP the value appearing in the diagramme is zero.

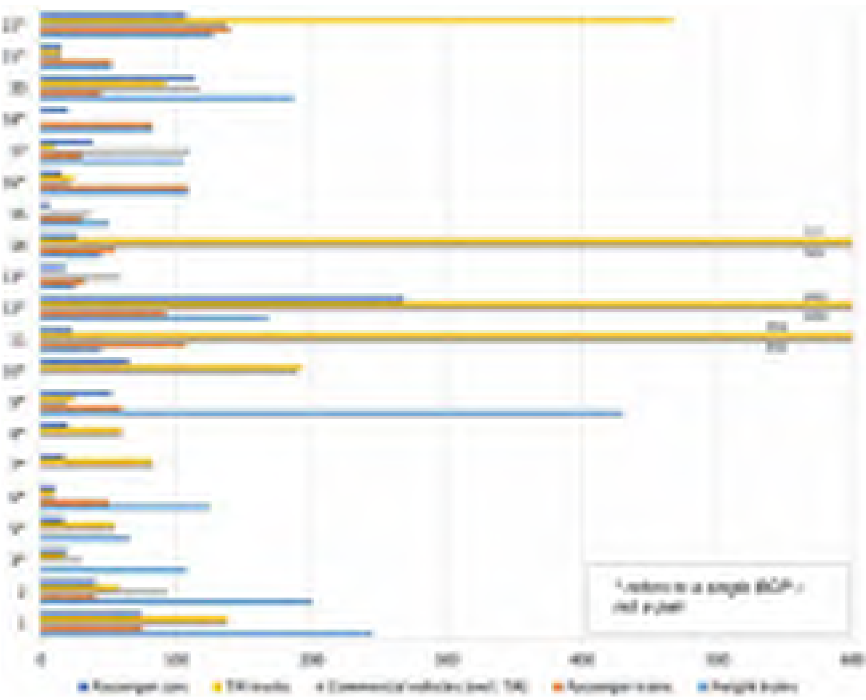

Figure 4 Total (waiting and procedural) transit times for crossing each pair of Road and Rail BCPs (average in both directions, in minutes)

As expected, commercial vehicles and freight trains face the most significant problems in terms of waiting and procedural times required. The reason for this is the inexistence of appropriate equipment and the insufficiency of working staff (in terms of number and not effectiveness). At Rail BCPs the main reason of delays is the change of locomotive and the unavailability of locomotives in the neighboring country, which usually is not ensured for hours. The following figures present the comparison of data obtained from the BCPs users. Figure 5 presents the average transit times reported by rail operators for five BCPs with the values reported by the respective Rail BCPs authorities. Times reported by operators are much higher than those reported by BCPs authorities (this does not apply only at one BCP).

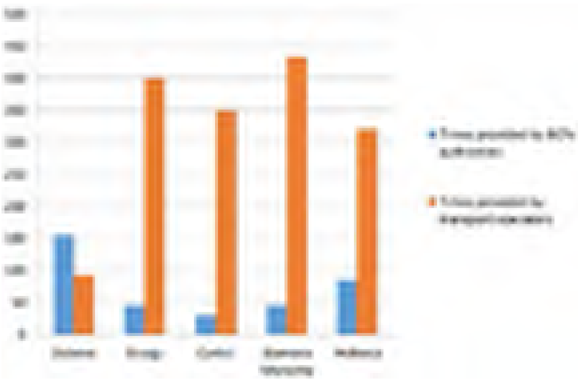

Figure 5 Comparison of total (waiting and procedural) transit times reported by users and BCPs authorities at 5 single Rail BCPs (average in both directions, in minutes)

Figure 6 presents a comparison of the transit times reported by BPCs authorities with the respective reported by truck drivers. It clearly shows the difference on how the users and BCPs authorities perceive the times spent at borders, even though users could exaggerate. However, there are several cases, for which the total transit times reported by the BCPs authorities oversubscribed the respective time reported by the truck drivers. Only two are the cases for which the times reported both by the BCPs authorities and the truck drivers were similar. The reasons for these differences in the times reported in order to be fully understood must be linked with the problems reported at each BCP both by the authorities and the truck drivers (in the majority of the cases they both have the same perception in this issue) and be fully analysed case by case, which exceeds the purpose of the current paper.

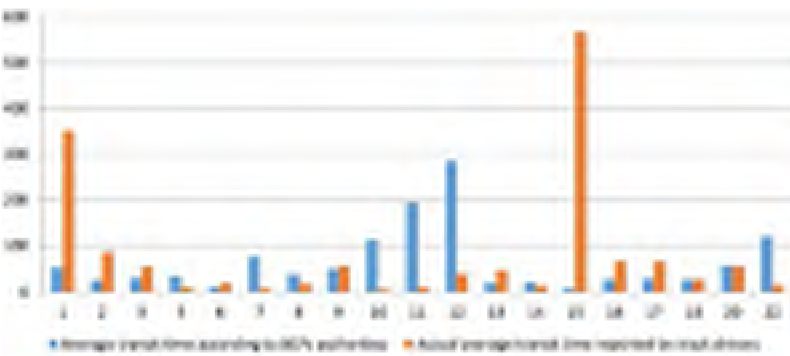

Figure 6 Comparison of total (waiting and procedural) transit times reported by truck drivers and BCPs authorities (average in both directions, in minutes)

Figure 7 presents a comparison of the transit times required at some BCPs according to the ACROSSEE survey with times reported by truck drivers and have been extracted from diagrammes of the International Road Transport Union (IRU) during various periods in the past, and thus the comparison is indicative. However, it clearly presents the difference on how the users and BCPs authorities perceive the times spent at borders, even though users could exaggerate.

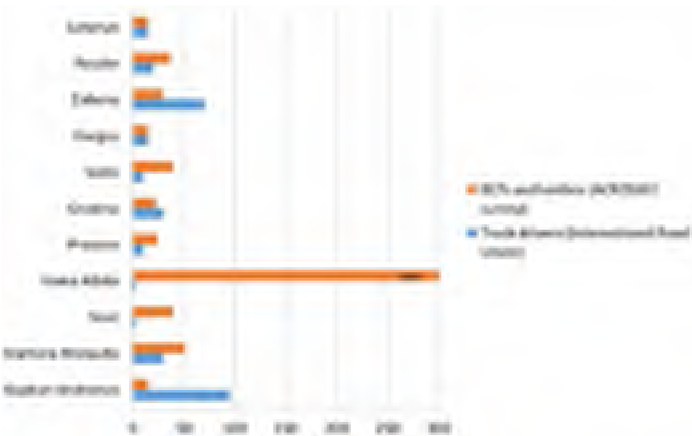

Figure 7 Comparison of total (waiting and procedural) transit times reported by truck drivers to IRU and by BCPs authorities at 11 single Road BCPs (average in both directions, both for TIR / non-TIR trucks, in minutes)

4 Conclusions

As experience shows from previous similar surveys, many problems could emerge, mainly due to a) the nature of the subject, i.e. regarding sensitivity of data related to borders that lead to unwillingness of some BCPs authorities to provide data or actual information and b) the large geographical coverage and the simultaneous performance of the surveys that make the supervision a very hard task, if not impossible (at least for the developer of the surveys' methodology).

The BCPs surveyed (presented in bold in Table 1) might not represent all the BCPs of the region, but certainly represent the majority of them along the most important international routes. Moreover, the biggest part of the surveys were conducted before the accession of Croatia in the EU, and thus the effects on BCPs operation, due to the displacement of the EU external borders, are not reflected in this analysis.

The knowledge of the current situation and the identification of the problems are the cornerstone for providing solutions. This applies also to the specific case of BCPs in the framework of ACROSSEE. Even having to face specific difficulties due to the particularities of the subject and – mainly – due to its sensitivity that additionally hampers data collection, the volume and quality of the data collected are considered satisfactory for the scope of the project.

More thorough analysis of the BCPs and, after the transport model is established, the representation of the main international transport corridors in time-distance diagrammes with the presentation of all bottlenecks at BCPs, other nodes or links of the road and rail networks (also in terms of capacity shortage) are expected to highlight the importance of the time lost at each BCP.

The results of the surveys, despite any weaknesses, can present the problem of border crossings at its real dimension, and could be a mean for highlighting the importance of measures

at BCPs, versus large-scale infrastructural investments, which require substantial maturity, financing and implementation time. These measures could bring equal or more benefits in terms of travel time reduction, minimal risks for environmental degradation, within shorter period of time and with submultiple investment costs.
ACROSSEE results in general could affect decision making, by enhancing the argumentation of the permanently affected and potentially benefited communities (importers, exporters, transport operators, chambers, etc.) against the politically driven large-scale projects, which are differentiated many times according to the switching of persons in the governments and Ministries of the various countries.
The facilitation of transport of persons and goods should be a permanent point of national agendas. The framework offered by the EU to its member states, as well as the efforts of the non-EU countries towards the integrated border management according to international best practices that are promoted by the international organisations, comprise a promising framework, which is anticipated to be further enhanced very soon, with the establishment of the Transport Community Treaty in South East Europe (between the EU and Western Balkan countries).

References

[1] Miltiadou, M., Basbas, S., Mintsis, G., Taxiltaris, C. & Tsakiropoulou, A.: Transport infrastructures in South East Europe. The planning experience and perspectives in the context of the European Transport Policy", 2nd International Conference on Road and Rail Infrastructure (CETRA 2012), Department of Transportation Faculty of Civil Engineering, University of Zagreb, 7 – 9 May 2012, Dubrovnik, edited by Stjepan Lakušić, ISBN 978-953-6272-50-1, pp. 203-212.

[2] SEE/A/071/3.3/X "South Eastern European Transport Axis Cooperation" (SEETAC) Project, Transnational Cooperation Programme South East Europe, Mobility report in the SEETAC study area, 2012.

[3] European Union: Regulation No 1315/2013 of the European Parliament and of the Council of 11 December 2013 on Union guidelines for the development of the trans-European transport network and repealing Decision No 661/2010/EU.

[4] SEE/D/0093/3.3/X "Accessibility improved at border CROSsings for the integration of South East Europe" (ACROSSEE) Project, Transnational Cooperation Programme South East Europe, Shared methodology for common standards analysis on border crossing points, 2013.

[5] Miltiadou, M., Taxiltaris, C., Mintsis, G. & Basbas, S.: Border crossings along the Pan-European Corridor X: Infrastructural and procedural improvements and derived benefits, 12th World Conference on Transport Research (WCTR '10), Lisbon, 11 – 15 July 2010, Proceedings paper ID 2783, pp. 1-25.

[6] International Road Transport Union, Border Waiting Times Observatory, http://www.iru.org/bwt-app.

Anđeli interchange on Matulji – Učka section of Adriatic higway (B8)

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Abstract

Matulji – Lupoglav section of the Adriatic highway was opened in 1981. and after 30 years it is in dire need of upgrade. Currently, construction of the second carriageway on this section is in design stage, but there were also upgrades on the existing section done, especially in the last couple of years. This was necessary because of the increase of traffic, but also because of large increase in residential construction on the outskirts of the highway corridor, and sometimes even in the corridor. In 2006, BINA Istra as the concessionaire of the Istrian Y, which Matulji-Učka is a part of, decided on three main points of upgrade on the existing section. These were construction of the merging lanes on the existing Veprinac interchange (Phase 1), construction of the new Anđeli interchange (Phase 2) and construction of the 2km long slow vehicles lane from Frančići viaduct to Anđeli viaduct (Phase 3). After conceptual design of the proposed upgrades, which explored multiple variants and options, final decision was made by the concessionaire that on Veprinac interchange merging lanes for vehicles connecting onto the highway in both Učka and Matulji direction should be designed, new Anđeli intersection will be designed to increase traffic safety and slow vehicles lane will be designed on the mountain side of the existing carriageway to allow for retaining of traffic throughout the construction period. This paper will focus on the design and construction of the new Anđeli interchange, from the conceptual design that included multiple variant options in 2006 to finalization of the project and opening of the interchange for use in the summer of 2012.

Keywords: infrastructure, road design, highway, interchange design, upgrade

1 Introduction

Even after construction of slow vehicles lane on the 1.5 km stretch of B8 in front of “Učka” tunnel entrance at the Kvarner side, ther were still issues with traffic congestion, especially in the summer months. Also, existing emergency stop areas near “Anđeli” viaduct were used by the locals as an illegal road entry point, and existing Veprinac interchange was built with direct exits from county road 5048 towards Učka and Matulji, without any merge lanes. Because of that, this part of the B8 was considered lacking in traffic safety, and it was decided to improve the existing state even before the construction of the second carriageway.
Conceptual design of existing section improvements along with solutions for construction of additional 1.8km long slow vehicles lane and merge lanes in Veprinac interchange, produced multiple variants of the future “Anđeli” interchange. Finally, 5 different variants were proposed to the Investor, which differed in layout disposition of the interchange, and in type (semi-interchange with only turn-out lane from Matulji and merge lane towards Matulji, or a full interchange). After revising the proposed variants, the Investor decided that a full interchange should be designed, with two design constraints. First was that the existing frame structure that served as a road passage under the B8 highway must be kept, so traffic can be

maintained throughout the construction period, and the second was keeping of the existing water reservoir in the future interchange.

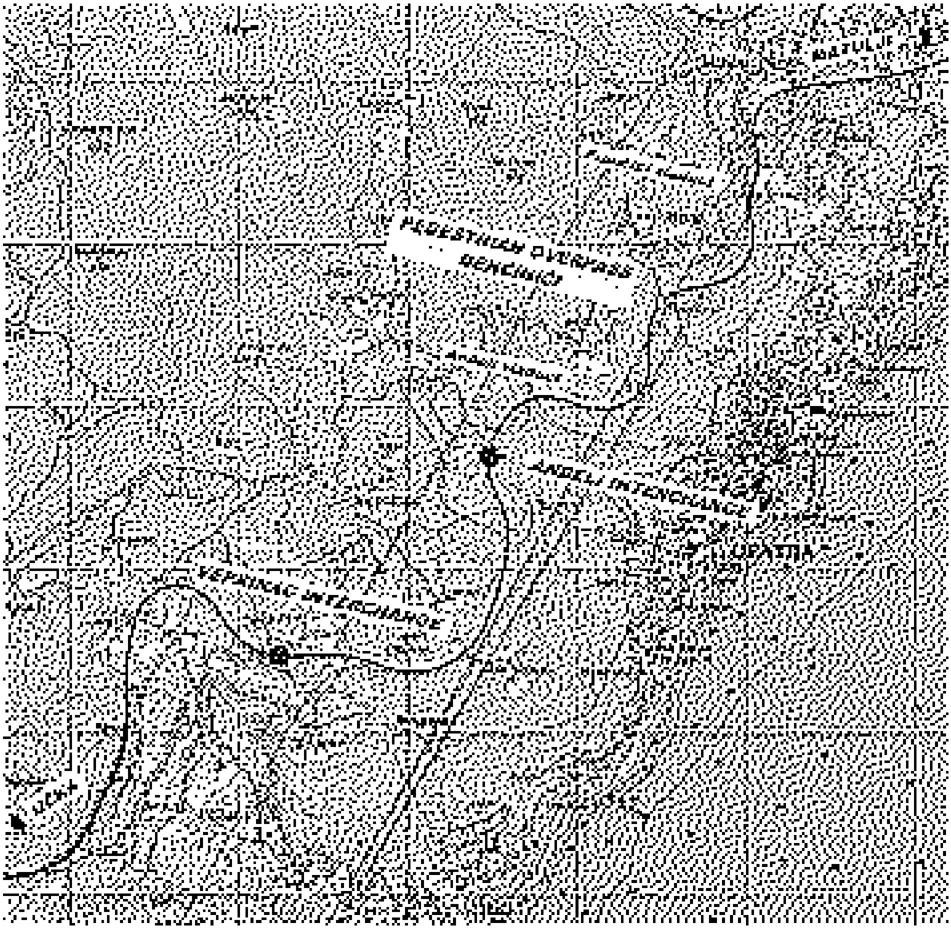

Figure 1 Matulji-Učka section of Adriatic highway (B8)

2 Design stage

2.1 Interchange

Anđeli junction is serving primarily as a connection to settlements of Kolavići and Zatka, but in the future, city of Opatija plans to build a local road network that it is an essential part of. Given the terrain configuration, the interchange was designed as a half-clover stretched over a long section of main road, but in a narrow corridor, to minimize the earthworks. Steep terrain slopes also imposed usage of minimum design elements.

Figure 2 “Anđeli” interchange

Technical elements of the interchange allow for design vehicle speed of 30 km/h on ramps, while the speed on the main road is limited to 80 km/h. Also, because the existing road underpass has inadequate width of only 4 meters, alternate traffic will be kept until the construction of second highway carriageway.

Table 1 Technical elements used for interchange design

minimum horizontal radius	25 m
maximum slope level (ramps)	10 %
maximum slope level (other)	12 %
lane width (traffic/emergency)	3.25/2.5 m
shoulder width	2.0/1.5 m
crossfall (straight/curve)	2.5/max. 7.0 %
minimum vertical radius (crest/sag)	250/200 m

Connection to the existing road to Kolavići was reconstructed and moved to surpass the level difference in excess of 12m at an acceptable longitudinal slope of 10.8%. Also, because of the level differences, design of retaining walls was necessary. Total of 5 retaining walls were designed, 2 on the northern side to protect the Slatina flood stream, and 3 on the southern side for protection of the existing water reservoir / cutoff chamber “Kolavići”. Since the geotechnical investigation works showed that terrain consists mostly of rock, cuts were designed with slopes between 1:1 and 2:1, with maintenance shoulders on cuts higher that 8m. Fill embankment slopes were designed at 1:1.5. Since the merge lane towards Matulji required the demolition of existing emergency stop area, it was moved app. 240m in direction of Matulji. Because of this, emergency phones installation and optic fibre cables in highway shoulders were also moved.

2.2 Viaduct

Since the existing road underpass, which is a concrete frame structure, couldn’t be reconstructed, the new structure that serves the merge lane in Učka direction was designed separate from it in every way. To accomplish this, the new structure has pile foundations, to also minimize influence on existing B8 embankments. In constructive sense, viaduct is a single span slab propped on head beams with variable height – a monolithic structure.

Table 2 Viaduct technical elements

span	12.8 m
width	7.10 m
beam height	1.50-4.65 m
pile diameter	0.8 m

Structure was calculated in SOFiSTiK software, with finite elements method, as a spatial construction elastically propped on the ground. Dimensioning was carried out for Ultimate limit state (ULS), and checked for Serviceability limit state (SLS).

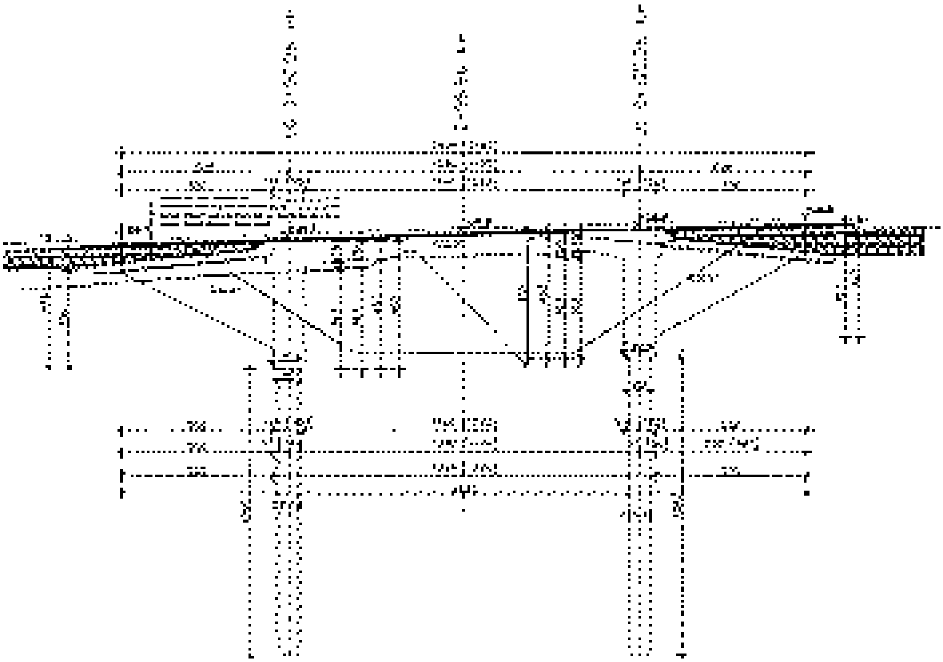

Figure 3 "Anđeli" viaduct longitudinal profile

2.3 Permits obtaining procedure

Long period of time needed for finalization of this project was in no small part a direct consequence of prolonged legal procedures. Preliminary design was started in October of 2008, and in July of 2009 the request for Location permit was submitted. It wasn't until May of 2011 that the Location permit became valid. In the 2 years that passed, procedure was delayed by many factors, including transfer of jurisdiction, land owner appeals and bureaucracy issues. This was cause for subsequent postponing of slow vehicles lane construction since financing for that part of the project was terminated because of the delays.
Fortunately, construction permit obtaining procedure was much faster and without big problems. This meant that construction permit was issued in November of 2011, which allowed the construction to start in early 2012.

3 Construction

Construction of the interchange started in February of 2012, as soon as weather conditions allowed. Deadline for opening of the new interchange was set for the beginning of the tourist season. It was carried out by Bouygues TP as a main contractor, and Cesta Pula and other local companies as subcontractors. The new road viaduct construction works were carried out by Viadukt.

3.1 Changes in original design during construction

In the process of construction preparation, it became obvious that the company that manages local water and drainage has no need for the existing water reservoir / cutoff chamber "Kolavići". This information didn't emerge in the design stage because of jurisdiction issues. Since it became clear that the reservoir can be demolished, two of the five designed retaining walls lost their purpose and were substituted with embankments. Instead of the existing reservoir, pressure reduction station was built along with a new pipeline. Also, after negotiations with Croatian waters agency, which is responsible for all waterways in Croatia, two retaining walls designed to protect the Slatina flood stream were substituted with a series of concrete culverts. This meant that only the retaining wall on the connection road to Kolavići was constructed, and main and construction designs were updated accordingly.

Figure 4 Slatina flood stream bed and culvert entrance

During the construction, design of cuts was changed to accommodate wider maintenance paths with access on both beginning and end of the cut. This required changes in the plot division reports, which was done according to as built documentation and survey.

3.2 Construction finalization and interchange opening

Construction of the interchange was finished in a predicted timetable, with only mild obstructions to ongoing traffic. Beginning of July 2012, internal technical audit was performed by the Ministry of Internal Affairs and Ministry of Maritime Affairs, Transport and Infrastructure. This allowed for opening of the interchange even before the official technical audit. The official audit was performed in late July, and with it, construction of the new Anđeli interchange was officially over after six years of design and construction.
In 18 months after the interchange opening, there was only one significant accident, caused by a driver who performed an illegal U-turn at the emergency stop area position. Fortunately, there were no fatalities, only material damages. This proves that the reasons for construction of "Anđeli" interchange were valid.
Currently, local population is requesting that City of Opatija continues with improvements to road network that are a part of the local spatial plans. Construction of the "Anđeli" interchange allowed the planned connections to Benčinići and Slavići to move from wishful thinking into possible projects. Time will show if this is just the beginning of improvements or a lonely exception.

References

[1] Hidroelektra-projekt d.o.o.: Jadranska autocesta (B8), dionica Matulji-Učka, Poboljšanja na postojećoj dionici, idejno rješenje, Zagreb, 2006.

[2] Hidroelektra-projekt d.o.o.: Jadranska autocesta (B8), dionica Matulji-Učka, Poboljšanja na postojećoj dionici, idejni projekt, Zagreb, 2009.

[3] Hidroelektra-projekt d.o.o.: Jadranska autocesta (B8), dionica Matulji-Učka, Poboljšanja na postojećoj dionici, dio građevine: Čvorište Anđeli, glavni projekt, Zagreb, 2010.

[4] Nadilo, B.: Dosadašnja izgradnja I dovršetak prve faze Istarskog ipsilona, Građevinar 58 (2006), pp 307-316, 2006.

[5] www.bina-istra.hr

[6] http://www.novilist.hr/Vijesti/Crna-kronika/Opatijac-prezivio-slijetanje-opel-astre-s-ceste-za-tunel-Ucku-u-40-metarsku-provaliju

INVESTMENT PLAN FOR BAR – BOLJARE MOTORWAY

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Abstract

Nowadays the main transport project in Montenegro is Bar-Boljare motorway, the new infrastructure project. The 170km new motorway will connect Serbia to the north of Montenegro, and further south to the Adriatic Coast, and with the Port of Bar, as a major port in the Adriatic. This motorway will be part of Bar-Belgrade-Budapest European Corridor, linking Montenegro to Central Europe, and presenting the transport project with very high national and regional priority. After the main sections of the motorway have been defined the Montenegrin government should make final investment plans for realization in the future. The authors developed the multicriteria model using the Analytic Network Process – ANP, as a solution for analysing and ranking 5 sections of the Bar-Boljare motorway. The network structure of the problem leads to the application of the ANP.

Keywords: investment plan, transport projects, analytic network process

1 Introduction

Project Bar-Boljare motorway is a key element in the strategy of the Montenegro Government of accession to the European Union, as it will allow Montenegro to be fully integrated within Europe. The project is also very important for the unification of the country as it will allow the north-east regions to be connected to the coast through our capital. Finally, the project will allow our key port of Bar to be fully connected to the rest of the European corridors and better serve Serbia and Kosovo, further facilitating the unlocking of this part of the Western Balkans and contributing to economic and political stability in the region.
Motorway toll-road proposed for linking the Adriatic coast at Bar via the capital Podgorica to the Serbian border at Boljare. Planned to connect Montenegro with Republic of Serbia through Požega – Belgrade and further link on the TEN-T corridor X, and hence to Romania and Central Europe. It would also connect with routes to the regional capital cities of Sarajevo in Bosnia and Herzegovina, Tirana in Albania and Skopje in Macedonia, therefore Bar-Boljare motorway has a clear strategic role to play in the region. The approximate length of this link is of about 170 km. Since the size of Montenegrin economy and the estimated total investment value of Bar-Boljare motorway project, which exceeds 2 billion, it's evident that this motorway corridor has to be divided into sections, which all together form an entity, from a technological point of view. After defining the five relevant sections of this motorway corridor, the program of their mutually synchronization in time and space should be set up. In accordance with that, the topic of this paper is defining the final rank of considered sections of the motorway corridor in Montenegro, considering the relevant set of criteria, subcriteria and interest groups.
The authors suggest using the multicriteria decision making approach, the Analytic Network Process, to define the investment plan for Bar-Boljare motorway.
This paper is organized as follows. After the Introduction, the following section is dedicated to the model description. All system's elements: alternatives, criteria, subcriteria and stake-

holders are named. The third section, the brief description of the ANP approach, shows the main steps of the applied multicriteria approach. The next section is Results and discussion, presenting the final obtained results of the model. Finally, the last section contains concluding remarks and future researches.

2 Model description

Following the main concept of the ANP approach, to have different clusters, mutually connected, with or without feedbacks, etc. The developed model has 6 clusters: alternatives, 4 groups of criteria with subcriteria and stakeholders.
All system's elements are presented in the figure 1.

Figure 1 The developed model

2.1 Alternatives

In the existing planning and project documentation, motorway Bar-Boljare corridor is defined as follows: Bar – Djurmani – Sozina tunnel – Virpazar – Tanki Rt – Farmaci (Podgorica) – Smokovac (Podgorica) – Mateševo – Andrijevica – Berane – Boljare (border with Serbia). This was the base for defining the five considered sections, table 1.

Table 1 Sections of the considered motorway

Section		Length
A1	Bar (Djurmani) – Virpazar	11.7 km
A2	Virpazar – Smokovac	38.2 km
A3	Smokovac – Mateševo	43.5 km
A4	Mateševo – Berane	34.3 km
A5	Berane – Boljare	41.3 km
TOTAL:		169 km

In corridor Bar-Boljare motorway, section Djurmani – Sozina tunnel – Virpazar, approximately of 10 km of semi-motorway has been constructed, within 4,2 km of Sozina tunnel, as well as temporary linkages with existing roads in Sutomore and Virpazar. It is proposed that the Bar-Boljare motorway and planned Adriatic-Ionian motorway have a common alignment in zone of capital Podgorica, in the length of approx. 10 km. Proposed motorway sections have been

coded as dual-2 links (2 lane in each direction). Within the model, the motorway has been given the following characteristics:
· lane in each direction;
· design speed of 100 kilometers per hour;
· capacity of 30 000 vehicles per day per direction.

Based on experts' opinion relevant criteria and subcriteria for the model are defined (table 2). Some of them are mutually connected. For instance, the criterion "Increasing the enterprises' competitiveness" is in relation with the criterion "Contribution to the regional development", etc.

Table 2 Criteria and subcriteria of the model

Criteria	Subcriteria	Description
C1 Costs	C11	Construction costs
	C12	Maintenance costs
	C13	Operating vehicle costs
	C14	The economic rate of return EIRR
	C15	Period of construction
C2 Traffic	C21	Number of accidents
	C22	Traffic volume
	C23	Alternative routes
	C24	Forecasted traffic volume
	C25	Changes of traffic flows
	C26	Infrastructure capacity utilization
C3 Environmental impacts	C31	Environmental protection
	C32	External influences
	C33	Demographic changes
C4 Benefits	C41	Travel time savings
	C42	Attractiveness of investment
	C43	Contribution to the regional development
	C44	Increasing of the security
	C45	The impact to the regional significance
	C46	Valorization of the potential
	C47	Tourism development
	C48	Easier access to market
	C49	Area development
	C410	Increasing the enterprises' competitiveness

2.2 Stakeholders

As the relevant stakeholders, following six interest groups are considered:
· S1 – Government;
· S2 – Local authorities;
· S3 – Construction sector;
· S4 – Tourist sector;
· S5 – Private sector;
· S6 – International financial institutions.

Their relative importance is defined, so the final rank among them is: Government, International financial institutions, Construction sector, Tourist sector, Private sector and Local authorities, respectively.

3 Brief description of the ANP approach

The ANP approach has been widely used for developing the model as a support system in the decision making process. The model with network structure is very good for presenting the nature of the problem in practice. The first step in this approach is developing the pairwise comparison matrices, presenting the priority among elements, using the fundamental Saaty scale [7] (table 3).

Table 3 Fundamental Saaty scale

The importance	Definition
1	Equal
2	Intermediate
3	Moderate importance
4	Intermediate
5	Strong importance
6	Intermediate
7	Very strong importance
8	Intermediate
9	Extreme importance

The matrix "A" shows a comparison among elements a_{ij}, representing the experts' priority of one element over the others. The matrix "M" is normalized matrix "A" with elements a'_{ij}.

$$A = \begin{array}{c} \\ A_1 \\ A_2 \\ \\ A_j \\ A_n \end{array} \begin{array}{c} \begin{array}{cccccc} A_1 & A_2 & \cdots & A_i & \cdots & A_n \end{array} \\ \begin{pmatrix} 1 & a_{12} & \cdots & a_{1i} & \cdots & a_{1n} \\ a_{21} & 1 & & & & a_{2n} \\ & & & & & \\ a_{j1} & a_{j2} & & 1 & & a_{jn} \\ a_{n1} & a_{n2} & \cdots & a_{ni} & \cdots & 1 \end{pmatrix} \end{array} \quad , \quad a_{ji} = 1/a_{ij} \tag{1}$$

$$M = \begin{bmatrix} a'_{11} & a'_{12} & \cdots & a'_{1n} \\ a'_{21} & & & \\ & & & \\ \cdots & & a'_{ij} & \cdots \\ a'_{n1} & \cdots & & a'_{nn1} \end{bmatrix} ; a'_{ij} = a_{ij} / \sum_{i=1}^{n} a_{ij} \tag{2}$$

The vector of priorities, "W", is an eigenvector of the matrix "A". The factor λ_{max}, where n is a number of criteria, is used for calculation of the consistency index of a matrix of comparisons, CI. This is the main advantage of the eigenvector method.

$$W = \begin{bmatrix} w_1 \\ w_i \\ w_n \end{bmatrix}; \quad w_i = \frac{1}{n}\sum_{j=1}^{n} a_{ij}^{'} \tag{3}$$

$$\lambda_{max} = \sum_{i=1}^{n} \left(w_i \cdot \left(\sum_{i=1}^{n} a_{ij} \right) \right) \tag{4}$$

$$CI = (\lambda_{max} - n) / (n - 1) \tag{5}$$

After the consistency index is calculated, the consistency ratio, CR, can be considered as a relation of the consistency index and the random index, RI. For CR › 0.1, the degree of consistency is satisfactory. Otherwise, the judgment of a decision maker should be revised.

$$CR = CI / RI \tag{6}$$

Table 4 The values of RI

n	1	2	3	4	5	6	7	8	9	10
RI	0	0	0.58	0.9	1.12	1.24	1.32	1.41	1.45	1.49

For calculating the final rank of alternatives, the normalized super matrix and the limit matrix should be developed. These calculations can be done in the software Super Decisions (www.superdecisions.com).

4 Results and discussion

After applying the developed model for ranking the sections of the Bar-Boljare motorway, the obtained results are presented in the following table.

Table 5 Final rank of considered projects

Section		**Rank**
A3	Smokovac – Mateševo	1
A1	Bar (Djurmani) – Virpazar	2
A2	Virpazar – Smokovac	3
A4	Mateševo – Berane	4
A5	Berane – Boljare	5

The section A3, Smokovac-Mateševo, will make better links between the north and south of the country. With a better appreciation of the potential in the field of economy and tourism development in the northern region, it will increase the accessibility to the hardly accessible regions, increase mobility, change market conditions and increase competitiveness of enterprises. Also, this project will increase employment and change the structure of employment, with involvement of local constructions firms, equipment, materials and labor in the construction phase, which will have its multiplier effect on indirect benefits, it will bring significant benefits that go far beyond the economic and financial benefits.

Southern sections A1, Bar-Virpazar, and A2, Virpazar-Smokovac, are highly ranked related to economic and social benefits, especially considering traffic demand. These sections have the highest value of traffic demand.
Section A4, Mateševo-Berane, with constructed section A3, Smokovac-Mateševo, leads to even better valorization of the potential of the northern region, better connection to the main road, regional and local roads, increasing benefits related to the travel time savings, vehicle operating costs, increasing the level of security, etc.
Section A5, Berane-Boljare, is the border section of the Republic of Serbia, which attractiveness could become even higher due to the fact that some sections of the Belgrade-Požega have been already designed and built.

5 Conclusions

The main transport infrastructure project in Montenegro is Bar-Boljare motorway, as a part of Bar-Belgrade-Budapest European Corridor. This road is divided into 5 sections, which should be ranked for investment in the future. This paper presents the model for ranking these sections using the Analytic Network Process, the multi-criteria approach. The model has alternatives, criteria and subcriteria, as well as stakeholders, all together make the network with mutual links and relations. The final obtained rank is as follows: Smokovac-Mateševo, Bar-Virpazar, Virpazar-Smokovac, Mateševo-Berane and Berane-Boljare.

References

[1] Caliskan, N.: A decision support approach for the evaluation of transport investment alternatives, European Journal of Operational Research, vol. 175, issue 3, pp. 1696-1704, 2006.

[2] Dikmen, I., Birgonul, M. T. and Ozorhon, B.: Project appraisal and selection using the analytic network process, Canadian Journal of Civil Engineering, 34, pp. 786-792, 2007.

[3] Ivanović, I., Grujičić, D., Macura, D., Jović, J., Bojović, N.: One approach for road transport project selection, Transport Policy, Vol. 25, No. 6, pp. 22-29, 2013.

[4] Macura, D., Bojović, N., Nuhodžić, R., Šelmić, M., Bošković, B.: Evaluation of transport projects using multi-criteria decision making method, International Conference on Traffic and Transport Engineering ICTTTE, Belgrade, November 29th to 30th, 2012.

[5] Macura, D., Bošković, B., Bojović, N., Milenković, M.: A model for prioritization of rail infrastructure projects using ANP, International Journal of Transport Economics, Vol. XXXVIII, No. 3, pp. 265-289, 2011.

[6] Macura, D., Nuhodžić, R., Bojović, N., Knežević, N.: One model for rail infrastructure projects selection, 2nd International Conference on Road and Rail Infrastructure – CETRA 2012, 7–9 May 2012, Dubrovnik, Croatia, pp. 533-538, 2012.

[7] Saaty T.: Decision Making with Dependence and Feedback: The Analytic Network Process, RWS Publications, 1996.

[8] Saaty, T.: Fundamentals of Decision Making and Priority Theory with the Analytic Hierarchy Process, RWS Publications, 2007.

[9] Tsamboulas, D., Yiotis, G. and Panou, K. D.: Use of a multicriteria methods for assessment of transport projects, Journal of Transportation Engineering, 125 (5), pp. 407-414, 1999.

[10] Tsamboulas, D., Yiotis, G. and Mikroudis, G.: A method for multi-criteria analysis in transportation infrastructure investments, International Journal of Transport Economics, Vol. 1, 2007.

[11] Tudela, A., Akiki, N. and Cisternas, R.: Comparing the output of cost-benefit and multi-criteria analysis – An application to urban transport investments, Transportation Research Part A, 40, pp. 414-423, 2006.

PROBLEMS TRACING BYPASS CORRIDOR IN SMALL CITY IN THE EXAMPLE OF DRNIŠ

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Abstract

Experience analysis in developing concept bypass of small cities shows that the implementation of new transport infrastructure projects of high rank is associated with significant problems that result in the extension of the deadlines of the projects. This paper describes the issues of regional plans as a basis for designing roads. It points to the lack of an accurate analysis of the planned corridor. Also the insufficient number of current data and information in spatial planning documents that the designer used during the execution of the project is discussed. The need to create additional solutions for corridors in order to create the optimum balance between benefits, costs and impacts based on the evaluation of the traffic model and the spatial position solutions is emphasized. This paper shows a way of finding solutions for corridor bypass in small cities enabling sustainable development of these cities in the future.

Keywords: *road design, spatial planning, transport planning, evaluation and selection of solutions*

1 Introduction

To create design solutions tracing corridors of major infrastructure facilities, collaboration of road designers, spatial planners, environmental experts and local government, is very important from the beginning of the design. This type of collaboration in recent practice was not commonly applied.
Problems tracing corridor bypass in small towns is shown in the example of the bypass of Drniš. The position of the city of Drniš in the existing road network is shown in Figure 1. In dynamics of development and integration of the Šibensko-kninska County, in coast-hinterland relation, the need for the construction of the Drniš bypass became obvious. This need was already observed already when developing technical documentation study to build a fast road Šibenik – Drniš – Knin – BiH border (BC). The existing transport network of Drniš is made of old roads with bed technical elements, which are not suitable to support traffic that would be directed between the planned high-speed road through the intersection "Drniš" and state roads D33 and D56. The route of the planned Drniš bypass starts in the area of the village Siverić on the state road D33, before crossing the railroad Knin-Šibenik, and it finishes as an extension of the connecting road of intersection "Drniš" and highways in the intersection "Drniš" in BC.
Planned road network of Drniš bypass consists of two corridors, one of which is provided under the spatial planning documents, and the other is evaluated through "Study to determine the optimum solutions of Drniš Bypass", and can be described as follows:
- CORRIDOR I – BC Tromilja-BH (ŽC6246) – St. John Cemetery – Badanj – D33;
- CORRIDOR II – BC Tromilja-BH (ŽC6246) – St. John Cemetery – passage under an industrial platform-planned bond D33 and D56.

Figure 1 Position of the city of Drniš in the existing road network

2 Project background

2.1 Spatial planning documentation

Until recently, spatial plans were made without detailed analysis of their impact on the surrounding area and the environment, and taken that the planned activities are often several decades old, and that the area and its content has changed, the result is very often a need to plan a route out of the planned corridor. The result is a growing number of modifications of the route that leads to the alteration of spatial plans, which is a time-consuming process. This way of designing and collaboration requires great effort and significantly extends the deadlines given by investors, and is a common situation with facilities of line infrastructure that spatial plans wait for optimized design solutions. The project background for designing the corridor of Drniš Bypass consists of the following documentation:
- Spatial Plan of the Šibensko-kninska County, modification and addition, August 2012;
- Physical Plan Drniš- modification and addition, Urbing d.o.o., April 2006;
- Urban development plan of Drniš – modification and addition, Urbing d.o.o., 2009;
- Conceptual design of a fast road Sibenik-Knin-Drnis-BiH border, shares Tromilja-BiH border, Faculty of Civil Engineering, Zagreb, November 2009;
- A study to determine the optimum solution of building a fast road Tromilja-Drniš-Knin-Strmica, Faculty of Civil Engineering, Zagreb, in December 2010;
- A study to determine the optimum solutions Bypass Drniš, Faculty of Civil Engineering, Zagreb, December 2012.

The Conceptual design of the bypass Drniš under the current spatial planning documentation is designed in CORRIDOR I. The plan from spatial planning documentation to build the corridor was abandoned in the way so that the bypass could continue to the connecting road of inter-

section "Drniš" of fast road Šibenik-Knin-Drniš-BiH border, which prevented the construction of another bridge over the Čikola river. Modification and addition of spatial plan of Šibensko-kninska County in August 2012 have fully accepted the general design of the CORRIDOR I, on the basis that the spatial planning documents of a lower rank needs to be updated.
In the process of environmental impact of Drniš Bypass, during the public consultation the proposed Conceptual design of CORRIDOR I was rated as unacceptable by the citizens of Drniš. The citizens of Drniš expressed their disagreement with constructing the bypass route in the area of State Road D33 to St. John Cemetery, where the planned route of bypass divides the Badanj village in two parts.
Newly designed bypass route (CORRIDOR II) takes over the existing network traffic from the state road (D33) which passes through the southern part of the city and opens up new approaches to the town from the north. The northern area of the city was planned to be a commercial and residential zone. UDP Drniš is in the process of modification, and through this procedure it is necessary to implement the redevelopment of area. The bypass contributes to the development of the planned zone and allows heavy freight traffic access without loading the town (street) network. Diverting transit traffic on the bypass unloads the approach roads to the town, and in the east the road traffic no longer intersects the railway traffic. Figure 2 shows an overview of variants of the previously described corridors of Drniš bypass in one of the existing spatial planning documents.

Figure 2 Urban development plan of Drniš with a preliminary ruling bypass

2.2 Geodetic background

In order to develop technical solutions of road systems a common procedure was performed by creating a digital elevation model for the project, as follows:
· scan HOK 1:5 000;
· vectorization of contour plan, height and structure characteristics;
· calculation of triangulation.

For the project the following documents were obtained: TK100, TK25 and digital orthophoto plans that were used for technical solutions and their spatial valuation.

3 The methodological approach

The methodology for selection of optimal variants consists of the following basic steps:
· analysis of spatial planning documents;
· development of technical solutions planned road network;
· development of transport models;
· definition of network scenarios;
· calculation of costs and valuation scenarios.

3.1 Traffic modelling

The traffic model is the basis for determining the traffic impacts of specific scenario which is analysed in the following section of the study. For the purposes of the preliminary design of the Drniš bypass a traffic model is made for road traffic in 2011 and in 2030, for narrower and wider areas in which the traffic impacts of road transport without investment and traffic impacts with the investment are calculated. The calculation of the effects of traffic is made for the following components:
· operating cost;
· travel times;
· accidental cost;
· pollutant volume;
· CO2 volume.

The existing transport network is shown in Figure 3. Statistical data on migration shows that the active population mostly perform their work (60%) within the settlement they live in, while 20% travel between settlements of Drniš. 20% of the active population commute to work out of town. Data obtained by counting the number of transactions that are carried out at several locations in the city, as well as data on counting traffic on national roads were used for calibration and evaluation of models. Projected traffic in the year 2030 was conducted with traffic growth of 2%. Figure 4 shows review of the planned traffic network load in 2030.

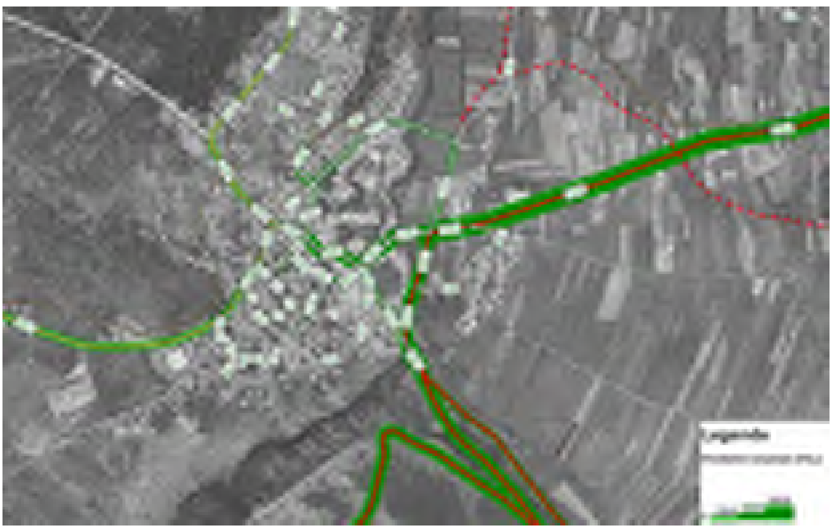

Figure 3 The existing traffic network of Drniš

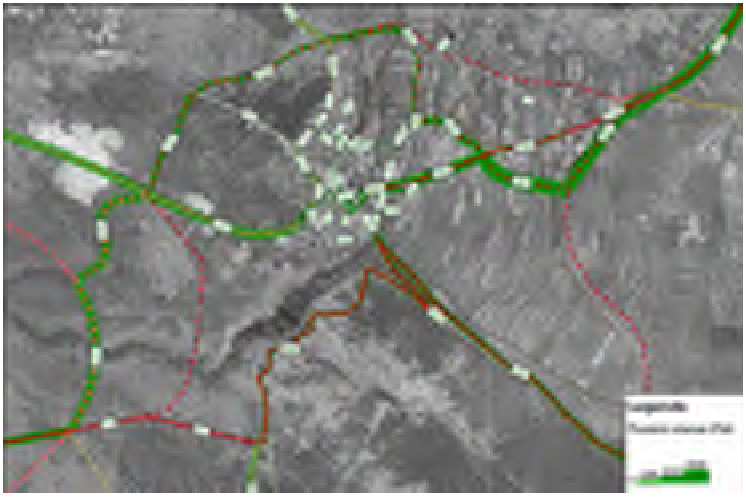

Figure 4 Planned network traffic load in 2030.god.

3.2 Description of segments and presentation of technical solutions

The bypass of Drniš is intended as a road with a pavement with two traffic lanes and the design speed of 80 km/h. Drniš bypass road segments are shown in Figure 5. Road network of the Drniš Bypass is divided into seven (7) road segments:

- a – IDRJ bypass Drniš – June 2011, CORRIDOR I;
- e – BC Tromilja-BiH (ŽC6246) – connecting road for Drniš;
- f – connecting road for Drniš – D33;
- g – D33 – D33 planned bond and D56;
- h – planned connection D33 and D56;
- i – planned connection D33 and Drniš bypass;
- j – connecting road for Drniš.

Figure 5 Overview of road segments

3.3 The calculation of costs and evaluating scenarios

By the combination of road segments for the planned network developed five (5) scenarios. For each scenario construction costs and traffic impacts are calculated. Review of scenarios, their calculated costs and economic indicators, is given in Table 1. Basic construction costs are calculated for all road segments as follows:
· route;
· objects;
· drainage.

Economic indicators are calculated for total costs (investment and maintenance) and transport effects, as follows:
· savings;
· Net Present Value – NPV;
· Benefit / cost ratio – B / C;
· Internal Rate of Return – IRR.

Net present value is calculated for the period of operation of 20 years with a discount rate of 5%.

4 Selection of optimal solutions and evaluation of variant

Economic efficiency is based on a comparison of costs and benefits (cost – benefit analysis) actions taken in the transport system (planned event – to do something) with the costs if the planned action is not taken (maintaining the status quo – do the minimum). Based on the basic parameters of the economic efficiency of the net present value (NPV), cost-benefit factor (B/C) and internal rate of return (IRR) acceptability of all proposed scenarios in road network system of Drniš is evident. Table 1 which provides an overview of the costs and economic indicators show that all scenarios offer positive values, which is an indication that all scenarios are thoroughly acceptable.

Table 1 Overview of costs and economic indicators

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

By comparing scenario 1, which represents Drniš Bypass in CORRIDOR I and scenario 5 representing the Drniš Bypass in CORRIDOR II, it is evident that bypass scenario in CORRIDOR II has better economic indicators.

Scenarios 2 and 3 represent parts of the Drniš Bypass, while Scenario 4 is an addition to the network in terms of defining the entire road system of the City of Drniš that makes Drniš bypass and connection of existing D33 and D56. Economic indicators of previously described scenarios also show the acceptability of these solutions, and the actual construction of the Bypass in CORRIDOR II proved to be very flexible solution.

Scenario 5 as the solution of the Bypass in CORRIDOR II according to criterion of savings, NPV, B/C and IRR is in the second place, right after the scenario 4 that represents its supplement on connection of D33 and D56. If you consider the ratio of investment per kilometre scenario 5 is the most cost-effective scenario of the Drniš Bypass. In conclusion, it is determined that scenario 5 as the solution of the bypass in CORRIDOR II, if we compare all the technical and economic indicators, is the optimal solution for the bypass.

5 Conclusion

The analysis of practice experiences shows the importance of cooperation between all stakeholders (including representatives of regional and local communities) from the start of design services. Spatial planning documentation that provides a starting point for road designing generally is not precise enough and does not contain all current data and information that designers need to consider when creating conceptual designs. This often results in smaller or larger deviations of finally designed route from the corridor outlined in the spatial planning documentations. Therefore, we can expect more frequent occurrence of the physical plans to adopt design solutions verified through the traffic, economic and spatial aspect, whose analysis should be carried out in the earlier stages of the strategic development of transport infrastructure projects. In this way the significant and increasingly frequent prolonging the period of completion of design and study activities would be avoided.

References

[1] Idejno rješenje obilaznice Drniša, Građevinski fakultet Zagreb, Zagreb, siječanj 2013.

[2] Studija izbora optimalnog rješenja obilaznice Grada Drniša, Građevinski fakultet Zagreb, Zagreb, prosinac 2012.

[3] Koren, Ž., Lugić, E., Satinović, V., Stepan, Ž., Rigo, A.: Problematika izrade studija utjecaja na okoliš projekata prometne infrastrukture, Dani prometnica 2013 – Mjerenja, ispitivanja i monitoring na prometnicama, Zagreb 2013., str. 407-427

[4] Prostorni plan Šibensko – Kninske županije, izmjene i dopune, kolovoz 2012.

[5] Prostorni plan uređenja Grada Drniša-izmjene i dopune, Urbing d.o.o., travanj 2006.

[6] Urbanistički plan uređenja Grada Drniša-izmjene i dopune, Urbing d.o.o.

IMPORTANCE OF TEMPORARY TRAFFIC REGULATION DURING CONSTRUCTION OR RECONSTRUCTION OF ROADS

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Abstract

During construction work on individual road network sections it is necessary, for shorter or longer period of time, to close for traffic some small part or even the entire length of the section. At the same time, unobstructed traffic should be ensured redirecting to the nearby road routes or, if no such routes exist, constructing temporary (or permanent) road sections and intersections is needed. Traffic redirection during the construction or reconstruction of high priority roads, with heavy traffic volume, is a problem which is additionally enhanced in cases of main city routes.
An example of such problem is the South Osijek Bypass, a part of the D2 Croatian state road. With traffic load of 18,500 vehicles/day AADT (Average Annual Daily Traffic), this is one of the busiest roads in Croatia. The role of South Osijek Bypass is traffic redirection outside the city and connection of major roads to Našice (D2) and Valpovo (D36) to the west, Vukovar (D2) and Erdut (D213) to the east, Beli Manastir (D7) in the north and Đakovo (D7) and Vinkovci (D518) to the south. Northern pavement of South Osijek Bypass was built between 1973 and 1989 year with two traffic lanes intended for two-way traffic while the southern pavement extension was postponed because of, at that point, insufficient traffic volume. Southern pavement construction started in year 2011, during which extension of four intersections and building of a whole one new intersection was needed. Given that the old, northern pavement was constantly under heavy traffic, the solution of temporary traffic regulation during the works was a particular challenge for the designer.
The paper describes traffic solutions for unobstructed traffic flow during construction works on the southern pavement extension of South Osijek Bypass with special emphasis on the importance of quality traffic flows diverting.

Keywords: temporary traffic regulation, reconstruction of road, South Osijek Bypass, heavy traffic volume, traffic flow regulation

1 Introduction

During construction or reconstruction of roads, special attention should be paid on uninterrupted traffic flow throughout the whole construction period. The uninterrupted flow of traffic can be secured by placing the appropriate traffic signs (a simpler way, in normal traffic conditions) or special, particular regulation of traffic (more complex manner, in particular traffic conditions) [1].
Particular traffic regulations, by their definition, represents all the measures to ensure the uninterrupted traffic flow at some stretch of the road. Particular traffic regulation is necessary in cases of complex construction; such as work at intersections, construction of the half-width of the road, in the course of alternating one-way traffic, etc.

Sometimes is necessary, for shorter or longer period of time, to close for transport small stretch of road or even the entire length of the road. With simultaneous progress of the construction, road users should allow an uninterrupted ride on nearby roads or, if no such directions, performing a temporary (or permanent) road sections and intersections. About all the measures for traffic regulation, it is necessary to inform promptly all road users about the new traffic regulation in a given period.

1.1 Legal and technical regulations

The obligations of investors and contractors before and during the execution of works and the mode of organisation of temporary traffic regulation during the construction or reconstruction of roads are determined by the legal and technical regulations [2-6].

Act of Roads [2], defines the obligations of the legal entity that manages roads during construction works. According to Art.62, Act requires establishment of appropriate temporary traffic regulation during construction. Traffic regulation must be established for ensures safe traffic flow and unhindered performance of work or other activities.

Act of Road Traffic Safety [3] defines a mode of marking construction works and obstacles on the road. Art.10 requires visible marking of road (or part of road) where works are carried out. Marking must ensure road users safety. Part of the road is marked by setting the appropriate traffic signs [4], and the safety of traffic participants is provided by placing bumpers.

According to the General Technical Requirements for Road Works, Volume I, [5], the contractor is required to set up and maintain traffic signs on construction site and in all prescribed places in the required number, form and technical characteristics in accordance with the work progress and the requirements of legally relevant institutions. The Contractor shall procure and maintain temporary lighting for the road, which must provide the same level of light as public lighting, which it replaces.

Temporary traffic diversion, according to GTR [5], will be performed in areas where buildings or construction site intersects existing roads, intersections, pedestrian and bicycle paths, or parts thereof, and because of the safety or scope of work it is not possible to establish a satisfactory traffic flow.

In the case of buildings affecting the traffic flow in the wider region and a number of associated existing roads, as well as in the case of national roads, the investor is required to ensure project development of temporary traffic regulations and to obtain all necessary approvals from the relevant institutions.

The contractor is required for quality solution of roads for temporarily traffic. Solution has to be in accordance with the category of roads and with the application of relevant standards. The contractor has to ensure safe traffic during the construction works. Upon completion of the works the contractor shall remove the temporary roads [5].

Regulation of justified cases and the procedure of closing a public road [6] prescribes the events in which public road can be closed and the closing procedure. Under the public road closure, it is considered the establishment of banning or restricting traffic for all or certain types of vehicles on certain public road or part of a public road on certain days or for a specified period.

Reasonable cases for the closure of a public road, according to the Regulation [6], is performing of routine and periodic maintenance works and reconstruction of public roads, as well as other works or activities on a public road that affect traffic safety or can not be done without closing the public road.

It is necessary to establish the conditions under which a public road will be closed, such as determining the optimal time of closure of public roads, determining of a public road on which traffic can be redirect, determining the conditions for the establishment of temporary traffic regulation; determining the time at which the road must be opened and brought to the original state and determining the mode of informing the public.

2 Temporary traffic regulation during construction of South Osijek Bypass

2.1 Importance of South Osijek Bypass for City of Osijek and the surrounding region

Traffic redirection during the construction or reconstruction of high priority roads with heavy traffic volume is a problem which is additionally enhanced in cases of main city routes. An example of such problem is the South Osijek Bypass, a part of the D2 Croatian state road. With traffic load of 18,500 vehicles/day AADT (Average Annual Daily Traffic), this is one of the busiest roads in Croatia. The role of South Osijek Bypass is traffic redirection outside the city and connection of major roads to Našice (D2) and Valpovo (D36) to the west, Vukovar (D2) and Erdut (D213) to the east, Beli Manastir (D7) to the north and Đakovo (D7) and Vinkovci (D518) to the south.

Figure 1 South Osijek Bypass, a part of the D2 Croatian state road [7]

Northern pavement of South Osijek Bypass was built between 1973 and 1989 year with two traffic lanes intended for two-way traffic while the southern pavement extension was postponed because of, at that point, insufficient traffic volume. Southern pavement construction started in year 2011, during which extension of four intersections and build of a whole one new intersection was needed. Considering that the old, northern pavement was constantly under heavy traffic, the solution of temporary traffic regulation during the works was a particular challenge for design engineers.

2.2 Description of project solution: road section between intersections Vinkovačka and Čepinska

The length of the four-lane section Southern bypass of Osijek city is 12.1 km long. It was planned to upgrade five existing intersections and the construction of a new intersection – "Vinkovačka". The task of the new intersection "Vinkovačka" was to relieve traffic between intersections "Čepinska" and "Trpimirova" also between streets Drinska and Divaltova. Project solution of road section between streets Vinkovačka and Čepinska (Figure 2), including the new project solution intersection "Vinkovačka" were represented the very challenging task on the route of the Southern Bypass. On the mentioned road section there are three structures:

1 Underpass "Filipovica", length L=41,6 m (new object (two objects) demolition of the existing embankment below the northern pavement);
2 Overpass "Bosutsko" over railroads, length L=127,50 m (a new object with the existing);
3 Underpass "Vinkovačka", length L=44,20 m (four-lane city roads, demolition of existing

and construction of new structure, partial demolition of the existing embankment below the northern pavement, construction of two objects).

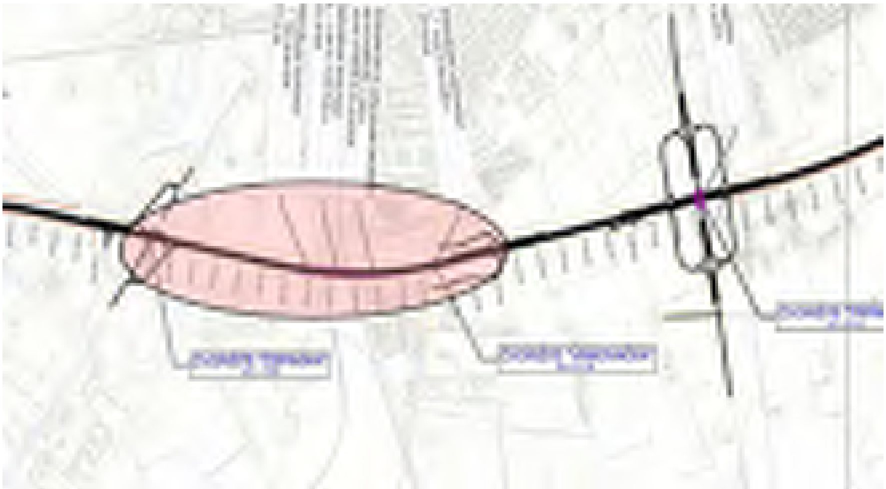

Figure 2 Part of layout drawing Southern bypass of Osijek: road section between intersections Vinkovačka and Čepinska [8]

2.3 Different variants selection of traffic regulation

Works on the embankment below the south pavement have led to problems with the uninterrupted flow of traffic. It was necessary to make a decision: will traffic on the north pavement during the constructions be stopped? Investor (Croatian Roads) and the Designer analyzed two variants:

1. Traffic to the northern pavement will not be interrupted during construction works. The existing embankment below the northern pavement would ensure with the planks in the length of 600 meters, installation of cement-gravel-clay piles (jet grouting), and with construction of AB diaphragm behind objects with a length of about 100 m. Estimated cost of this variant was 16.5 million HRK (2.1 million euros).
2. Traffic to the northern pavement will be stopped during the construction work and redirect to the bypass line of the existing road nearby to the agricultural economy “Ankin Dvor” (Fig.3.). Estimated cost of this variant, redirected traffic to the bypass line, amounted to about 6 million HRK (0.78 million euros).

Decision about the traffic conduction was analyzed as follows:

- costs of both variants;
- any unforeseen circumstances and constructions work that can further occur during the execution of works;
- possible traffic accidents and casualties passengers on the northern pavement due to open site.

Investor (company “Croatian Roads”), team of design engineers and representatives of the City of Osijek, made the decision to accept variant II as cost effective and safer traffic variant. With this variant all work on road section will be uninterrupted, without any potential traffic accidents, traffic jams, delays in operation at the site and without unforeseen works (and without additional financial resources) for the implementation of the most demanding part of the route South bypass of Osijek.

Figure 3 Final variant (II) of traffic regulation: bypass route of existing road [8]

In the main design [8] was planned closure of traffic on the northern pavement between intersections "Čepinska" and "Vinkovačka" with the traffic possibility by branches of the intersections loop. Diverting traffic from the north pavement between mentioned intersections was defined by the alternate route: bypass line of the existing road near by agricultural economy "Ankin dvor".
The bypass route (Fig. 3). is 4.5 km length and passes through the existing transport infrastructure of the city of Osijek. The route begins at the intersection "Čepinska" using ramp of intersection. After that, the traffic diverted to city road street St. L. B. Mandic in the direction of the roundabout at the entrance to the industrial zone. Further traffic passing industrial zone (Jablanova street) and cross the railroad tracks in the level and connects to the street Zapadno predgrađe. Then bypass route passes economy "Ankin dvor" and after crossing the railroad tracks at the level connects to Vinkovačka street to Vinkovačka intersection. Node "Vinkovačka" provides a connection of bypass traffic flow to the existing pavement of southern bypass and represents the end of the bypass route.
End of bypass route represented the biggest problem because it had to provide a crossing of two relatively strong traffic flow direction: one way the direction of the bypass route, and another direction from the city center. This is the busiest intersection during rush hour and, realistically, at this intersection was expected increase in traffic congestion.
Proposed problem solution and finally accepted, was roundabout (Fig.4). This solution, as the most rational and the safest, ultimately was derived. Traffic equipment and signaling on the road was organized to allow uninterrupted traffic on the bypass route. Velocity of traffic is regulated at 50 km/h.

Figure 4 Roundabout as solution for good connection of few traffic flows [9]

2.4 Traffic regulation in phased

Complete traffic regulation during construction works is made through two phases:
- Phase I – included upgrade of the existing road near by "Ankin dvor" in a length of 1.22 km. The existing road was 5-6 meters width, with surface of crashed stone, thickness of 50cm, in very poor condition. The existing road was upgraded from crushed stone material and a asphalt pavement.
- Phase II – included the construction of the northern and southern branches of nodes " Vinkovačka " (in accordance with existing project documentation [8]), the preparation of project documentation for the bypass route [9], and construction of controlled railway crossing with semi-barriers, light and sound.

The performance of first phase of bypass construction started end of January 2012 and during 2013. started final, second phase.

3 Conclusion

Before making the final decision about stopping and traffic diversion due to road construction should be analyzed the overall picture of traffic flow in the city and surroundings. The final solution can not be an overnight solution and should be approached from several directions, taking into considering all users in traffic flow. The contribution of the described traffic regulation, specially for community that lives along the road, is manifold: this solution is provided a safe traffic flow without congestion, construction works on the South bypass route will be unobstructed, according to the timetable, the local community got a new road as an alternative road and this solution is created the basic conditions for further development of the observed part of the city of Osijek. Finally, the leading idea, with selection of technical characteristic or economic optimization of road route, should be traffic safety. From the date of opening for traffic on the bypass direction weren't no major traffic accidents or delayed traffic flow.

Figure 5 The construction site on Southern bypass of Osijek: overpass "Bosutsko"

References

[1] Divić, A.: Cestovna signalizacija i sigurnost prometa, Priručnik za građenje, održavanje i rekonstrukciju cesta, Savez društava za ceste Hrvatske, Zagreb, 1976, 139-190.

[2] Zakon o cestama, Narodne novine br.84/11, 22/13, 54/13 i 148/13

[3] Zakona o sigurnosti prometa na cestama, Narodne novine, br.67/08, 48/10, 74/11 i 80/13

[4] Pravilnik o prometnim znakovima, signalizaciji i opremi na cestama, Narodne novine br. 33/05, 64/05, 155/05,14/11

[5] Opći tehnički uvjeti za radove na cestama, knjiga I, Hrvatske ceste i Hrvatske autoceste, Zagreb, 2001.

[6] Pravilnik o opravdanim slučajevima i postupku zatvaranja javne ceste, Narodne novine 119/2007

[7] www.google.com/maps/place/Osijek/ 10.02.2014

[8] Glavni građevinski projekt trase ceste; Južna obilaznica grada Osijeka, Dogradnja južnog kolnika, dionica „Koridor Vc-čvorište Tenjska“, Institut IGH d.d, svibanj 2011.

[9] Izvedbeni projekt obilaznog pravca za potrebe izgradnje državne ceste D2, dogradnja južnog kolnika južne obilaznice grada Osijeka, Institut IGH d.d., srpanj 2013.

NEW ROAD MAINTENANCE MODEL IN FINLAND – 2014 PILOT PROJECT

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Abstract

Finland is testing a new model for Performance Based Maintenance Contracting (PBMC) for roads in 2014. In Finland, PBMC have been used for about thirteen years and the market is very competitive, driven by lower costs, and quality needs improvement. Continued pressure towards efficiency and reduced costs are approaching the limits and a new model is needed to increase the quality and provide incentives for contractors. An internal road agency steering group was assigned the task to develop a new model for maintenance that would be appropriate for Finland. The "alliance model" has been tested in some countries, but there are limited results and the alliance model may not be appropriate for Finland. This new model is titled "Program Managed Performance-Based Maintenance Contracts" and has similarities to the alliance model, but is much simpler. Since the model is presently under development, the objective of the paper is to discuss the development, procurement, and preliminary findings. The tendering of this model is in early 2014 and details are not known at the time of the writing this abstract, but preliminary finding will be available for the conference. The new model introduces possibilities for improved quality, services, and provides a better opportunity for risk sharing.

Keywords: Performance Based Maintenance Contracts (PBMC), outsourcing, maintenance and contracting

1 Background Information

The Finnish Transport Agency (FTA) began operations at the beginning of 2010, where three important modes of transport, such as, road, rail and waterways were restructured into one transport agency. Previously, the road administration client was known as the Finnish Road Administration (Finnra), which managed the state public road network, but now FTA includes road, rail and waterways. FTA is responsible for maintaining and developing the transport systems as a whole in cooperation with other key stakeholders and its goal is to promote productivity and innovations in the transport sector. FTA is also responsible mobility of the transport system's network, environmental stewardship and responsible for operational steering of nine Centres for Economic Development, Transport and the Environment (known as the ELY Centres).
The Centres for Economic Development, Transport and the Environment were partially composed from the nine Finnish Road Administration regional offices that were also created during the reorganization of FTA in 2010. These nine ELY Centers have transport responsibility, which includes road transport mobility and safety functions on behalf of FTA's steering goals. In addition, the ELY Centers are responsible for maintenance and functionality of the road transport assets, systems, and promote safety and mobility of the road network. These ELY

Centers basically act as the "road manager" on behalf of FTA and is responsible for the procurement of capital investment projects and all outsourced maintenance contracts.

1.1 FTA and ELY-Centres routine maintenance contracts

In Finland, safety, mobility and availability of the road network is assured through meticulous maintenance. Road maintenance services are publicly tendered in the form of area-based contracts and currently there are 81 area-based contracts using the Performance Based Maintenance Contracts (PBMC) concept. The contractors preforming the road maintenance services are responsible to maintain the services according to the client's Levels of Service (LOS) for their respective road classification. In Finland, winter maintenance operations include snow plowing, prevention of slippery conditions or deicing of roads, repairing unevenness of the road surfaces, and ensuring the visibility of traffic control equipment, such as signs and road markings. In the summertime, the main duties include maintaining the gravel roads, traffic signs, vegetation control, grass and brush cutting, repairing and patching pavement defects, like potholes, and any other surface defects that may cause risks to traffic safety. The contractors are also responsible for continuous monitoring of the road system, availability to on-site tasks, reporting and tracking defects, and assisting authorities/officials when there are emergencies. The present maintenance contracts are shown in Figure 1 and also includes the location of the new pilot project that is being tested in Espoo.

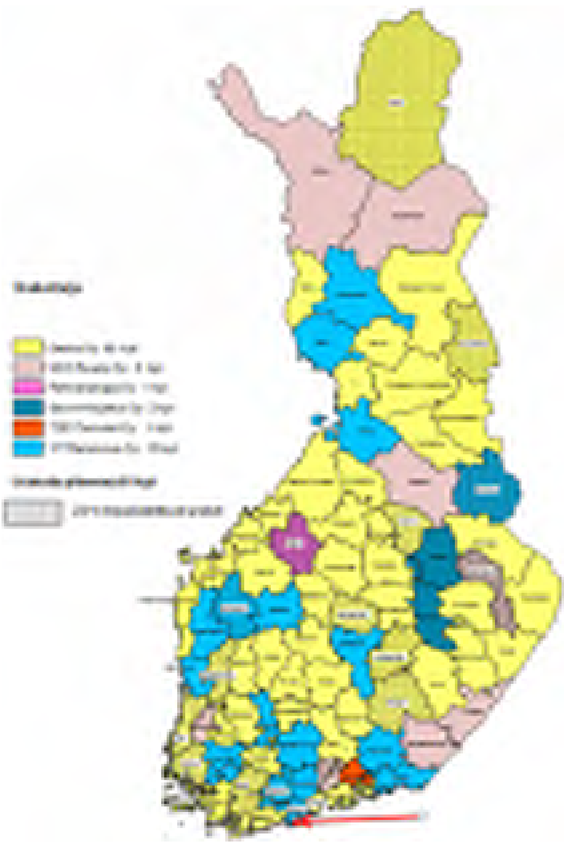

Figure 1 Area Maintenance Contracts in Finland.

2 Program Managed PBMC

The Finnish Transport Agency (FTA) initiated a feasibility study to evaluate the "Program Managed PBMC" model during the end of 2012. An internal working group with the assistance of experts from the infrastructure industry decided and recommended to pilot test the new model. After the feasibility study and preliminary development of the tender documentation, FTA and ELY Centre Uusimaa decided to continue the development of this new model in the fall of 2013. The new model is based upon principles and guidelines from the project management of maintenance contracts, alliance model, traditional routine and periodic maintenance contracts. The new model is termed "Program Managed PBMC" because the routine maintenance services significantly deviate from traditional project manager concepts and maintenance management concepts. A custom-made approach was necessary to achieve many new goals and desires of the client organization. At issue is a totally different management concept, where the procurement contracts are actually tendered and managed by the "contractor". However, the intent and spirit of the public procurement requirements should be observed and honored.

2.1 Client's goals

The new model is more appropriate for demanding and very demanding type maintenance contracts, where flexibility and allowance for variations are prominent, compared with traditional contracts. The model allows for flexibility during the contract period, client change orders can be included in the contract, and the potential for innovations and development. The main goal is to provide services from a road users' perspective and provide flexibility for implementation of these services. The model attempts to balance the risk and promote better teamwork and cooperation between the client and the contractor. The model is quite straightforward and is aligned with the present area maintenance contract models, which have the potential to be adaptable. The result should be less management and quality inspection.

2.2 Structure of the Program Managed PBMC

The provider/contractor (herein called the program coordinator) is responsible for the maintenance services for the area based maintenance contract according to the client's road classification and service levels. The service requirements of the program coordinator include many of the typical routine maintenance services, management and administration services and all related procurement duties.
The program coordinator offers professional and quality services on behalf of the client so that both parties have a common interest and focus for the maintenance services. The program coordinator together with the client should provide an effective implementation method and a cooperative management method that leads the entire maintenance process in a cooperative and open manner. The program coordinator is responsible for the maintenance planning process that needs to be approved and continually modified in a cooperative effort with the client. All purchases (product deliverables, performance, service requisitions and subcontracting) will be competitively tendered regardless of the number of bidders and even those that have provided alternative bidding prices. Maintenance activities are supported in the maintenance plans according to the procurement plans that are done in the name of the program coordinator. The client will nevertheless make final decision and approval.

2.3 Espoo Pilot Project

The area maintenance contract for Espoo is scheduled from 2014-2019 and is chosen as the pilot project for this new model. The new model is procured by Pirkanmaa ELY-Centre on behalf of Uusimaa ELY-Centre and is categorized as a very demanding maintenance contract, which includes the roads in the cities of Espoo, Kauniainen, Kirkkonummi, and outlined parts of Helsinki. The contract includes about 850 km of roads and 150 km of pedestrian and bicycle pathways. Winter road classification for this contract comprises about 130 km of dual lane roads of class 1S and about 200 km of single lane 1S, which includes ramps. The remaining road network is comprised of roads classified as II and III. The Espoo contract area is a highly travelled and often congested road network that is susceptible to disturbances throughout most of the day. Performing maintenance activities on the road network is challenging for the safety of both workers and road users, which requires significant attention and well planned activities and services. Another challenge for the maintenance contractor is the location along the coastal area, because the weather may change rapidly and temperatures fluctuate above and below the freezing point.

3 Procurement process

The procurement process for the pilot project is done in two stages. During the first stage, the official invitation for "expression of interest" was issued during the fall of 2013. The candidates expressing their interest in the project were required to have a turnover of at least €12 million and had previous experience in maintenance works or construction/project management. In addition, they were required to have two of their own staff that had a minimum of two year experience as project managers in routine or periodic maintenance contracts, providing their education, training and experience met the requirements. Also, there was a prequalification limit of five tenders.

All prequalified bidders were sent a preliminary proposal in the beginning of October 2013 and only the preliminary quality portion of the bidding documents were submitted to the client by the end of October 2013. Next, the client explained the initial contract proposal documentation through private individual negotiations for deliberation. These bilateral negotiations were intended to develop the invitation to tender documents, versus official contract negotiations.

The second stage of the procurement process was to gather lessons learned from the bilateral negotiations, finalize the invitation to tender documents and conclude when there is a signed agreement with the winning bidder.

The finalized invitation to tender documents was sent to those offers' participating in the negotiations at the end of January 2014. The proposal due date is planned for April 1, 2014 and the tenders are expected to be evaluated within a week, so that the client can have a signed contract with the award winner by the middle of May 2014.

3.1 Tender Evaluation procedure

The tender evaluation selection criteria established in this pilot project is using a competitive, negotiated method. The main focus of the award is one that is the most overall economically beneficial. The most overall economic beneficial tender is the one receiving the highest score from amongst the bidders. The procedure uses a 1000 point maximum scoring scale and is determined by the following weights of 20% for quality, 10% of key personnel and 70% for price.

The quality assessment category is evaluated by four different selection factors that are composed of the quality plans, program coordinator's procurement expertise, winter maintenance responsiveness, and the program coordinator's training and development. The scores from these four selection factors are multiplied by the appropriate weighting factors to determine the sub-category score and then summed to receive the quality category score.

The key personnel category is determined by the service and organization capability and by the examinations of key personnel. The evaluation includes the service and organizational capabilities and skill levels. The examinations seek to evaluate all the program coordinators, various key personnel expertise, and capabilities relating to the wide range of tasks and working requirements. The scoring of points is accomplished by averaging the total points scored form the tests. The scores from these two selection factors are multiplied by the appropriate weighting factors to determine the sub-category score and then summed to receive the key personnel's category score.

Figure 2 Components of the Target Price

The bid price is the "Target Price", which consists of all procurement related costs and acquisitions, management and administration, as well as the management service fee. The tender with the lowest target price will receive 10 points. The other bidders target price is compared with the lowest bidders target price. The price scores are multiplied by the appropriate weighting factor to determine the price score.

Table 1 Evaluation Scoring System

Category	Selection Factors	Max. poss. score	Weighting Factor	Weighted Score
A1. Quality of the Maintenance Management Plans	A.1.1 Evaluation of management plans, functional descriptions and initial quality points, and depot locations, inventory of accessories	10	9	90
	A.1.2 Program coordinator's procurement and philosophy	10	6	60
	A.1.3 Program coordinator's responsiveness and commitments, as well as management to the weather conditions	10	2	20
	A.1.4 Program coordinator's personnel training and development	10	3	30
	Subtotal of A.1		**20**	**200**
A2. Key Personnel	A.2.1 Service and organizational capacities	10	5	50
	A.2.2 Key Personnel Exams	10	5	50
	Subtotal of A.2		**10**	**100**
B. Bidders Price	B. Target Price	10	70	700
	Subtotal of B		**70**	**700**
Total Score			**100**	**1000**

The fixed management fee includes the program coordinator's general overhead costs, risks and profit margin. Contract management and administrative costs portion includes among other things; contract management and administrative costs, fulfillment of their duties, as well as office and information technology costs. The procurement costs include all related

purchases to fulfill the outcomes of the project requirements and includes; equipment purchases, all related sub-contracting, service contracts, construction and maintenance services and materials, temporary traffic devices and systems, and other related purchases, which are not included in the management or administrative cost portion.
The total score is calculated by a summation of all the points scored for all the sub-categories and the highest score is declared as the winner of the tender. Table 1 shows the details of the scoring system, categories, selection factors and the weights. Figure 2 shows the components that are included of the "Target Price".

3.2 Program Coordinator's Payment Mechanism

The annual target price includes all actual costs defined in the contract, such as management and administration costs, as well as the cost of purchasing. If the cost exceeds the target price, the client will pay the program coordinator 70% of the all costs exceeding the target price up to the guaranteed maximum price.
The guaranteed maximum price is fixed at 10% above the target price and can only be modified during the contract period through client ordered additional work, changes in the construction/maintenance index, external influences that exceed the guaranteed maximum price that are agreed by the client, or changes in the quality service levels. Otherwise, the program coordinator will be responsible for all costs exceeding the guaranteed maximum price.
The program coordinator has the option to transfer the costs exceeding the guaranteed maximum to the following year's procurement cost, providing that the client provides approval. The program coordinator is fully responsible for the prices in the last year of the contract for any costs exceeding the guaranteed maximum price.
Similarly, if the costs are below the target price the client pays the program coordinator an incentive of 30% of the amount below the target price up to a maximum of 250000 euros, in each contract year. If the incentive is exceeded in one year, the amount greater than 250000 euros can be transferred to the following years purchase price reduction. Figure 3 shows an example scenario of the payment mechanism.

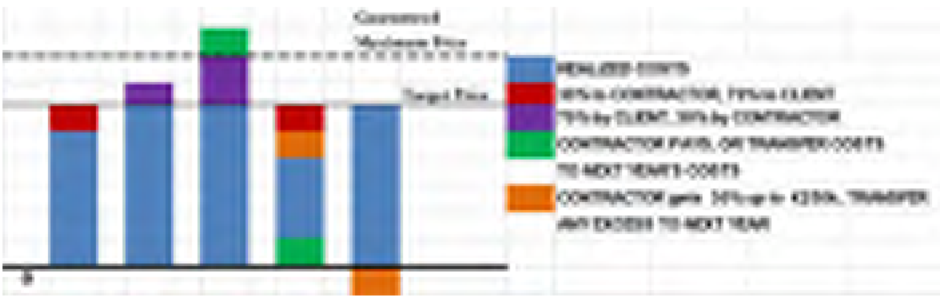

Figure 3 Example of payment possibilities

3.3 Additional potential incentives

There are four additional schemes or possibilities to receive incentives in this model, providing that the target price is not exceeded by the program coordinator. The first possible incentive is from the road user satisfaction and innovation. Traditional PBMC already include this incentive, so that is not new. The results from; the road user satisfaction survey during winter, a special evaluation team for measuring the performance during winter, improved summer services and innovations may receive a maximum incentive of 2% based upon the annual winter maintenance costs.

The other three incentive models have not been used in routine and periodic maintenance contracts. These three incentives are contract related, workers' and road user safety, and observations contributing to safety accidents. In second incentive scheme, the program coordinator may possibly receive an incentive of €15000 every year for providing good maintenance services, providing the program coordinator does not exceed the amount of penalty points in that year. If the program coordinator receives greater than 100 penalty points, then there is no incentive. Penalty points and the incentive calculations are shown below:

- written warning ⇢ 20 penalty points;
- A-Group Disincentive ⇢ 50 penalty points;
- B-Group Disincentive ⇢ 80 penalty points;
- B-Group Disincentive for not performing entire work on class 1 roads during winter ⇢ 100 penalty points;
- disincentive for not performing according to work plan #7 ⇢ 100 penalty points;
- dDisincentive for not performing according to the quality plans, follow-up reporting, and inaccurate data reporting from #8 ⇢ 100 penalty points;
- quantity of non-conformance report reminders are over 50% or equivalent errors in A-Group disincentives ⇢ 100 penalty points.

The incentive scoring structure:

- maximum incentive is 15000 euros, if there are no penalty points;
- 50 penalty points ⇢ incentive of 7500 euros;
- 100 penalty points or more ⇢ no incentive;
- intermediate values for the incentive is calculated using the formula: Incentive = 15000 × (1-b/100), where b is the period accumulated penalty points. The program coordinator is entitled to an incentive, if b ≤ 100.

The third incentive scheme consists of the program coordinator's observations of workers' and traffic safety hazards, deficiencies, problems in the contract area, and deficiencies from other operating companies maintaining other roads. The program coordinator may be entitled to a maximum incentive of €15000 by reporting observations, follow-up activities, new ideas for improvement work, and improved traffic safety measures. The incentive calculations are shown below:

- reporting 100 observations or ideas per year ⇢ an incentive of 15000 euros;
- reporting 50 observations or ideas per year ⇢ an incentive of 5000 euros;
- reporting under 50 observations or ideas per year ⇢ no incentive;
- intermediate values are calculated by linear interpolation.

The final incentive scheme is that the program coordinator may receive an annual maximum incentive of €20000 euros if road traffic accidents in the area contract are explained or resolved in greater than 60% of the occurrences. The incentive calculations are shown below:

- if greater than 60% of traffic accidents are explained ⇢ an incentive of €20000;
- if 30-59% of traffic accidents are explained ⇢ an incentive of €10000;
- if under 30% of traffic accidents are explained ⇢ no incentive;
- intermediate values are calculated by linear interpolation.

4 Discussion and remarks

The tendering phase is presently in process and there are a maximum of five bidders that are preparing their respective tender offers. The Espoo "Program Managed PBMC" pilot project is expected to be awarded in mid-May 2014, after the tenders are evaluated, and it appears that the way forward is proceeding satisfactorily.

The winning contract is expected to be determined in about two months and if all related procurement procedures can be completed in time, the contract can then commence on October 1, 2014. It is hopeful that the procurement model will work in practice, as well as, how the model appears from the development group members' perspective.

5 Conclusions

This paper presents a new model for the maintenance contract development in Finland and offers a potential model for others to consider. The Programed Managed PBMC is a totally new maintenance model that attempts to balance the risks and seeks to improve the quality, compared to the previous Finnish maintenance contracts. Since the procurement process is underway there are no actual results to date and the test of time will reveal if the results from this new model are acceptable. In theory, it appears to be adequate and until there are actual results can there be any valid conclusions.

References

[1] Levola, K.: "Program Managed Performance Based Maintenance Contracts, HUMPPA – Pilot Project in Espoo (2014-2019)", 30th Winter Road Congress in Finland, Jyväskylä, Finland, 21- 22 January 2014

EXPERIMENTAL SECTIONS IN THE HUNGARIAN ROAD MANAGEMENT

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Abstract

In the preparatory phase of new construction, rehabilitation and construction technologies there are several options to apply. The decision support can rely on computerised design programmes, laboratory test series, accelerated loading tests, monitoring of experimental (test) sections (and their eventual combination). These decision support means basically differ from each other from the view-point of time need and accuracy. This paper concentrates on the role and risks of experimental sections generally and in the Hungarian road management, highlighting some case studies. Sophisticated (professional) design, very careful construction, timely maintenance, as well as regular monitoring with expedient processing of condition data time series are all needed for the real success of an experimental section. If any of these preconditions is lacking, there is a real danger that misleading conclusions are (can be) drawn from the actual performance of the experimental section for the expected performance of the new construction or maintenance technology to be implemented. The Hungarian case studies to be outlined are connected with cement concrete pavements. Every case study is evaluated for its level of success; the eventual mistakes made there are also highlighted. Some general conclusions are drawn to provide information for the less experienced road sector stakeholders who plan to build trial section before implementing a promising new road technology in the near future.

Keywords: experimental section, road management, cement concrete pavement, continuously reinforced cement concrete pavement, whitetopping

1 Previous investigation of the suitability of innovation

It is evident that the continuous development and implementation of innovative materials and technologies can be considered as a must in every national economy sector. This statement is valid also for road engineering since the up-to-date road construction and maintenance techniques using innovative, high-performance materials are badly needed for meeting the challenges coming from the combined, synergic effect of ever increasing traffic and environmental load. However, the planned wide use of new materials and/or methods presupposes the proving of expected good performance as a reason for introducing the innovative material or technique instead of (or at least in addition to) the traditional “old” one. The typical investigation types for getting previous evidence about the suitability of the innovative procedure are: computerized performance models, laboratory test series, accelerated loading tests and trial (experimental) section monitoring [1].

1.1 Computerized performance models

Special computerized (e.g. finite element) models are available worldwide to forecast the performance of various pavement structure compositions. The expected deterioration of selected – eventually critical – condition parameters (stresses, strains, deformation etc.) as a result of the cumulative, synergic effect of mechanical and environmental loads can be obtained after relatively short running time as the output of the model. The real challenge for the engineer is the realistic selection of the programme inputs, the mechanical properties (e.g. E-modulus) of the pavement structural material in question. Similarly, it is a hard task to choose each characteristic deterioration mode to be considered in the calculation. This approach does not take much time but the level of accuracy in its results is rather limited due to the generalization and simplification of the inputs considered.

1.2 Laboratory test series

Another methodology for the forecast of the performance of an innovative material or technology is producing test specimens and carrying out special laboratory investigations. Typically, not only the samples made by innovative technologies are tested but also reference specimens are investigated in order to compare the performances. Even if the planned material composition is used in the laboratory test series, it is important to select the investigation methods which are more or less able to simulate the characteristic deterioration modes of the structure in question. The limitations of this relatively quick methodology are, among others, as follows: environmental loads cannot be considered at all or at least not realistically; the relatively small size of the laboratory samples tested results in different performance from that of the relevant built road element.

1.3 Accelerated laboratory tests

A lot of countries operate accelerated loading testing (ALT) facilities (circular tracks, linear tracks with load moving to and fro, etc.) which are usually open-air but a couple of them were made in buildings. In some cases, the artificial, (almost) continuous loading is performed by multi-axle, heavy trucks; however, it is more wide-spread that continuously moving, highly loaded wheels or axles are used for this purpose. Several technological variants – including reference one(s) – can be loaded simultaneously allowing a direct comparison of their performances. Another positive feature of this performance prediction procedure is that no previous "speculation" on the expected pavement deterioration modes is needed; the actual, experienced changing in the initial condition parameter levels provides information about the critical mode(s). However, some 10-year highway traffic load can be performed in ALTs in a month or two; the repeated environmental load of a highway section under "normal" traffic cannot be simulated when using accelerated loading facilities. (The ALTs in buildings can have the possibility to change air temperature, relative humidity, sub-grade moisture content, etc., but it does not allow realistic simulation). It is worthwhile to mention that building and operating accelerated loading facilities are rather expensive.

1.4 Trial (experimental) section monitoring

It is obvious that the construction and monitoring experimental (trial) sections can provide the most reliable forecast (prediction) about the performance of a new material or technology if, among others, the following important preconditions are met: careful selection of the location and sufficient length of experimental section(s); the Client's detailed disposition including the need for reference section; professional and thorough pavement design; well-equipped contractor motivated to innovation; effective quality management including continuous and tight independent quality control; careful planning of the monitoring of the trial and reference sections including the condition parameters investigated, measuring techniques (devices) to be used, durable marking of point-like measuring spots if any (in case of the use of non-continuous measuring techniques); selection of measuring frequency (typically every year); sophisticated evaluation of the condition data time series obtained for comparing the performances of innovative and traditional variants. Even if every precondition listed before are satisfied, and so, reliable performance prediction can be expected, the time-consuming nature of the methodology cannot be ignored.

2 Case studies of Hungarian road experimental sections

The long-term monitoring results of experimental sections (preferably together with neighbouring reference sections) can provide more certain and more reliable information about the appropriateness of the new material or technology than the other approaches.
Next two Hungarian case studies will be shown for road experimental sections pointing out their goals (the new technology to be demonstrated), the (eventual) reference sections, the frequency and period of monitoring, condition parameters to be monitored, measuring techniques applied, evaluation of the results and discussion of the actual level of success to have used the experimental section for assessing the performance of the innovative material or technology.

2.1 Case study on various cement concrete technologies

However, the first trials with cement concrete pavements were carried out in Hungary already in 1911, and a lot of them were constructed since, Ministry for Transportation decided to continue motorway construction programme using exclusively asphalt pavements from 1976 on. As a rather negative consequence, building of cement concrete pavements ceased on other – non-expressway – roads, as well. It was just in 1998 – following a 22-year break – that a ministry decision initiated the preparatory activities for restarting the cement concrete pavement construction in Hungary. KTI Institute for Transport Sciences, Budapest was commissioned to the preparatory activities [2]. The latest relevant foreign experience was gathered. The following three alternative technological variants were chosen for detailed investigation and the comparison of their actual performances:
- jointed and dowelled cement concrete pavement;
- jointed and dowelled cement concrete pavement with “exposed aggregate” surface;
- continuously reinforced cement concrete pavement.

After having evaluated the results of a detailed laboratory test series, the mix recipes were developed for the experimental sections. 35 to 42 MPa compressive strength range was selected with 0.40 to 0.42 w/c-ratios for each concrete mixture variant. (Plastificator was added in 0.10 to 0.12% of weight of water, and air-entraining agent in 0.04 to 0.08%).
As a site of the trial sections, a heavily trafficked – above 1100 heavy vehicle/day – secondary road (road 7538) close to the Slovenian state border was chosen. (In accordance with the Hungarian ministerial orders in 1999 – when trial sections were designed and constructed – no experiment was allowed to make on a main national public road). The road needed a complete pavement structure reconstruction due to its total failure. In addition to the three cement concrete experimental subsections with a length of 500m each, a neighbouring reference (control) subsection of 500m was also built using high modulus asphalt pavement with modified binder.

The pavement structure variants were as follows:
- experimental subsection 1:
  - 220 mm jointed and dowelled cement concrete pavement;
  - 150 mm mixed-in-plant cement stabilisation base course;
  - 100 mm mixed-in-place cement stabilisation sub-base.
- experimental subsection 2:
  - 220 mm jointed and dowelled cement concrete pavement with exposed aggregate surface;
  - 150 mm mixed-in-plant cement stabilisation base course;
  - 100 mm mixed-in-place cement stabilisation sub-base.
- experimental subsection 3:
  - 170 mm continuously reinforced cement concrete pavement (CRCP);
  - 150 mm mixed-in-plant cement stabilisation base course;
  - 100 mm mixed-in-place cement stabilisation sub-base.
- reference subsection:
  - 30 mm stone mastic asphalt with modified binder;
  - 80 mm asphalt binder course with modified binder;
  - 90 mm asphalt base course with modified binder;
  - 150 mm mixed-in-plant cement stabilisation base course;
  - 100 mm mixed-in-place cement stabilisation sub-base.

The construction of the trial sections was basically influenced by the extremely rainy spring and summer of 1999. That is why the design bearing capacity of 50 MPa at the surface of sandy gravel capping layer could not be reached, its stabilisation with cement was decided by the Client.
The trial subsections were built with 6m pavement width following the width of the secondary road. Just one of the traffic lanes was allowed to be closed during the construction. No continuously reinforced cement concrete pavement (CRCP) had been built in Hungary before the experimental section on road 7538. The relevant foreign literature was utilized in the design (thickness, dilatation structure, anchoring to prevent the horizontal end movement of CRCP, positioning of reinforcement etc.).
The principal goal of the experiment was to evaluate and to compare the performances of the four pavement structure types under "canalised" heavy traffic load. (The lorries run in very narrow wheel paths due to the 3 m traffic lane width).
The monitoring of the trial subsections was done first with a frequency of 6 months and later 12 months. The condition evaluation methodology all pavement condition parameters related to every possible failure type. Table 1 presents the failure types and condition parameters considered.

Table 1 Possible distress types and condition parameters measured on various subsections

Failure type	Condition parameter	Pavement		Spacing of evaluation
		Asphalt	Concrete	
Longitudinal deformation	unevenness	X		continuous
Slab faulting	unevenness		X	continuous
Rutting	transverse profile	X		3 fixed cross sections/subsection
Loss of skid resistance	skid resistance	X	X	50 m in alternate traffic lanes
Wear of aggregate grains	macro roughness	X	X	50 m in alternate traffic lanes
Surface defects	visual evaluation	X	X	continuous
Poor bearing capacity	bearing capacity	X	X	3 fixed cross sections/subsection
Pavement edge distress	pavement width	X	X	100 m

The location of measuring points was marked by durable painting on the pavement surface. The visual pavement survey resulted in a distress map with the location, type and severity of each surface defect detected. The first condition survey was performed in November 1999 to fix the initial (0) condition of the experimental section. The regular monitoring of the trial and reference sections has made it possible to identify the deterioration process of each condition parameter. Just some of the results are identified:

- already at the end of the first year, several punch-outs presented themselves on the continuously reinforced concrete trial part section; their further deterioration made it necessary to patch them using asphalt mixture to prevent the accidents coming from the sudden unevenness of the pavement surface; the subsequent investigation of the reasons of the very early deterioration has revealed the poor design (too thin pavement layer, too narrow pavement etc) as a decisive parameter;
- the tendency of surface roughness deterioration characterized by macro roughness and skid resistance (actually the combination of macro and micro texture of pavement surface) can be divided into 3 phases; in the first 2 years, an intensive deterioration is observed, the speed of this deterioration is significantly slows down in the period of 3-10 years of pavement age, while practically unchanged roughness values can be detected when the age of pavement exceeds 10 years; as an example, Table 2 presents some statistical parameters of the macro roughness and skid resistance values measured on the surface of jointed, dowelled cement concrete pavement with "traditional" surface at the age of 0, 2, 8 and 13 years (the pavement ages were selected of the almost continuous data time series for representing the three deterioration phases mentioned before). After 13 years of traffic load, both the texture depth and SRT-values deteriorate to nearly the same levels independently on the pavement and surface type selected, at the same time the standard deviations of the measuring data masses are much lower after high number of vehicle repetition number than immediately after the opening to traffic of the road with new pavement. The SRT-values of 50-55 and the texture depth ranges of 0.30-0.50 mm measured on the test cement concrete pavement surfaces after 13 years of traffic load with high heavy vehicle share are still appropriate (safe) values.

Table 2 Some statistical parameters of macro and micro roughness (measured using sand patch method and British pendulum) obtained at different ages of cement concrete pavement surface (test section road 7538)

Statistical parameters	Texture depth [mm]				SRT-value			
	at pavement age [year]							
	0	2	8	13	0	2	8	13
Mean value	0,99	0,67	0,52	0,40	79,4	63,4	60,7	51,9
Minimal value	0,70	0,42	0,37	0,31	66	53	58	50
Maximal value	1,27	1,16	0,72	0,51	90	72	65	54
Range	0,57	0,74	0,35	0,20	24	19	7	4
Standard deviation	0,17	0,17	0,08	0,07	6,55	4,56	1,78	1,29

2.2 Case study on whitetopping technologies

Laying thin cement concrete courses on asphalt pavements with repeatedly deformed surfaces is considered as a possible rehabilitation technique. When designing the first Hungarian whitetopping experimental section, the following technological features were considered:

- the thin cement concrete layers have 120-150 mm thickness, while the ultrathin ones are 50-120 mm thick;
- 20 years of life time can be expected on any old – asphalt or cement concrete – pavement if various technological preconditions are met;

- old pavement should have sufficient, homogeneous bearing capacity without fatigue cracks, rutting cannot exceed wearing course;
- concrete slabs are square;
- layers should be bonded to decrease bending stress in the structure;
- min. 75 mm remaining asphalt layer(s) thickness in the old pavement structure before using whitetopping is required;
- old pavement surface has to be sufficiently rough and clean for ensuring effective bond with thin cement concrete course;
- properties of whitetopping recipe: fibre dosage, high early strength;
- the layer is built without dowels and joint sealing;
- effective curing is a must.

A highly trafficked main road no.5 (daily 2750 heavy vehicles) at the outskirt of the city Szeged was selected as a site of the first Hungarian whitetopping experimental section [3]. The top layers of an extremely rutted, 7,0 m wide asphalt pavement section of 85 m length were milled, and thin cement concrete course was laid. The main elements of cement concrete recipe are:

· aggregate	1,832 kg/m^3;	· cement (CEM I 42.5 N)	420 kg/m^3;	· water	168 kg/m^3;
· plastic fibre	1 kg/m^3;			· plastificator	6,3 kg/m^3.

Compressive strength amounts to 27.2 N/mm^2 after 1 day, 39.3 N/mm^2 after 2 days and 43.1 N/mm^2 after 28 days. After 10 months of traffic load, 1.2 mm texture depth and 68 SRT-value were the average roughness values. The number of cracked slabs of 1.75 m x 1.75 m size reached 41, that is, almost 22 % of the 188 concrete slabs by 2013. Most of the cracks can be found in the outer lane, close to the two ends of the test section. Two of the worst slabs had to be replaced; otherwise local asphalt patching was performed. The reasons for the unexpectedly high percentage of early cracks are as follows:

- the actual heavy traffic load of the section was much higher than expected, among others, due to the many permissions given by Highway Authority for passing overloaded vehicles on the section;
- at the end of the experimental section, the heavy lorries reaching the first whitetopping slabs traveling from the connecting rutted asphalt pavement surface exerted high load causing their quick failure;
- at the other end of the section, a lot of multi-axle heavy road vehicles have to decelerate and stop because of the signal-controlled junction;
- manual laying resulted in the relatively low homogeneity of slab quality.

3 Concluding remarks

Experimental sections have a major role in the validation of new technologies and materials. However, they can provide reliable and useful data just if their design, construction, maintenance and operation are performed properly. Two Hungarian case studies are shown highlighting also the reasons of eventual improper performance. It could be concluded that the synergic effect of careful design, high-level construction (with competent and innovation devoted contractor) and the efficient independent quality control can be considered as important prediction for the success of road experimental sections.

References

[1] Gáspár, L., Horvát, L. & Lublóy, L.: Lifetime of transport facilities, UNIVERSITAS-Győr Non-profit Ltd., Győr, 2011, 324 p. (In Hungarian)

[2] Karsai, K. & Bors, T.: Construction and monitoring of experimental cement concrete pavements, Beton, 1, pp. 3-5, 2008. (In Hungarian)

[3] Boromisza, T.: Design of whitetopping, Transport construction Review, 11, pp. 32-34, 2010. (In Hungarian)

REDUCING COST OF INFRASTRUCTURE WORKS USING NEW TECHNOLOGIES

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Abstract

In the economic and financial conditions of our time the cost of infrastructure works is often the main factor upon which decide to whether or not to make an investment in the South – Eastern Europe. Another important factor to be taken into account in the development of infrastructure investments is the environmental impact of construction. Thus, since the design phase is focused on using new technologies, performing to ensure two objectives: low cost and low environmental impact. This paper aims to draw up possibilities to reduce costs by using new technologies both to the foundation layers and layers of asphalt mixtures. The focus is on the use of new materials and technologies that have a positive environmental impact by reducing both transportation and production costs. Since the design phase, based on laboratory studies, it is proposed the use of hydraulic binders for execution of road foundation layers, the use of asphalt mixtures with high stiffness modulus and warm mix technology.

Keywords: roads infrastructure, management, optimization, road structure design, asphalt mixture stiffness, fatigue

1 Introduction

In our days it can be seen a general increase in traffic flow and higher axle loads of trucks on Romanian roads which have steadily increased the demand for stronger and more durable road pavements. The increase in traffic flow also means that traffic interruption for maintenance becomes less desirable for roads user, since almost 60% of total amount of passangers and goods use roads network. The need to increase the service life of pavement submitted to traffic of growing magnitude imposes the use of asphalt mixtures with high performance. A solution that has been adopted in the last decades is the modification of conventional bitumen through the incorporation of polymers, enhancing fatigue resistance and reducing permanent deformations. With the purpose of extending pavement life, new technologies like high modulus asphalt mixes (MAMR), called "enrobé à module élevé" (EME), developed in France, have been developed.
A relatively new processes and products in Romania use various mechanical and chemical means to reduce the shear resistance of the mix at construction temperatures while reportedly maintaining or improving pavement performance. Warm-mix tehcnology is distinguished from other asphalt mixtures technologies by the temperature regimes at which they are produced and the strength and durability of the final product. Cold asphalt mixtures are manufactured at ambient temperatures, on the order of 20° to 50°C, while hot-mix asphalt is typically produced in the range of 150° to 180°C. Warm mixes are produced in the temperature range of 120° to 140°C. Hot-mix asphalt has higher stability and durability than cold-mix asphalt, which is why cold mix is used in the lower pavement layers of low-volume roadways. The goal with warm mix is to obtain a level of strength and durability that is equivalent to or better

than hot-mix asphalt. By conducting laboratory tests, the goal of this study is to highlight the properties of asphalt mixtures made with respect of these two technologies and to use laboratory datas (as stiffness modulus) in pavement design process. When it is about to design a pavement structure, there are few important factors to care about: initial cost, rate of degradation, effects of roads work by user delay cost and accident costs, environmental costs and state agency costs by maintenance cost and traffic management costs.

2 Benefits and problems associated with new technologies

2.1 Warm mix technology benefits

Warm mix technology has great benefits regarding the environmental protection and the improvement of paving operations. These benefits are described below.
Energy savings reported on WMA trials ranged from 20 to 35 % at the plant depending on the WMA system, moisture content of the aggregate and the type/efficiency of the plant. The energy savings may be equivalent to approximately 1.5 to 2.0 liters of fuel per tone of material.
Lowering the mixing and placement temperatures of bituminous mixes provides enormous gains in reduction of asphalt fumes. Visual observations of WMA during placement clearly show reduction in asphalt fumes and reports indicate reduction in the magnitude of 30 % while other more optimistic studies are indicating reductions of up to 90 % behind the paver.
Warm mix asphalt technologies facilitate compaction. Certain systems have been described as “Flow Improvers” to improve “compatibility” of bituminous mixes even in adverse windy and cold weather conditions. The objective of WMA systems is to modify the temperature/viscosity relationship in a manner such that, suitable mixing and compaction viscosities are achieved at lower temperatures, while adequate viscosity is maintained at service temperatures.
Warm mix asphalt facilitates transportation. Transportation constraints/distances may consist in a problem for placement of conventional HMA mixes. In a sense, transportation time consumes some of the HMA compaction time. This may become a major constructability issue when mixing and placement temperatures are relatively close. Increasing mixing temperature to compensate for long transportation is not viable as increased mixing temperature will likely damage the binder. Warm mix asphalt provides more flexibility; it is possible to increase mixing temperature above the “WMA mixing temperature” (but below the HMA mixing temperature) with limited binder damage.
With most WMA systems the temperature of the bituminous material at the end of compaction is lower than with HMA, and also closer to service temperature. Accordingly, the usage of WMA allows quick return to traffic. For the same reasons, multiple lifts of WMA may be place on top of one another within a short period of time. This is a net advantage whenever, deep fill of bituminous materials is required to be placed in a trench in a short period of time.

2.2 Warm mix technology possible problems

There is a general concern for WMA rutting performance that is connected with the decreased mixing temperature which may lead to incomplete drying of aggregates and insufficient coating with bitumen. Another aspect that may influence decreased resistance to permanent deformations is the decreased oxidative hardening of bitumen due to the lower production and compaction temperature. These problems might be treated with adding active adhesion agents or initially choosing harder bitumen grade [1].
Potential rutting problems require careful evaluation of asphalt in laboratory. Testing samples should be prepared carefully, because there might be a necessity for mixture aging before compaction to ensure proper correlation with actual production process. The choice of the

right compaction method might also be a problem, because some methods might not be sensitive enough to temperature changes.
Due to low mixing temperatures, moisture contained in the aggregate may not completely evaporate during mixing and the retained water in the aggregates could lead to increased susceptibility to moisture damage. Because of residual moisture left behind by the microscopic foaming process, this is even more critical for WMA technologies that involve foaming as a binder viscosity lowering action. These problems, if they occur, may be successfully treated with active adhesion agents.
WMA is reported to have better compaction potential due to decreased viscosity and less bitumen ageing in the production process. This can allow saving compaction energy and reducing the time necessary for compaction which may be especially important in low temperature paving. The reduced compaction risks, if realized, cause the cost that can far exceed additional costs for WMA production. However, if wax technologies are used, they require additional attention regarding the temperature conditions for rolling. The compaction must be finished before the wax starts to crystallize; after this temperature the wax forms lattice structure in the asphalt that may be damaged if the compaction is continued. This means that compaction window is shorter than for HMA and additional rollers may be required to reach the necessary density in the given time window [2].

2.3 High modulus asphalt mixtures technology benefits

Introduction of this concept is not new, it appears in 1980's, and, gradually, have prooved to main advantages. Improvement of asphalt mixture layers performances by a stiffer behavior, a better fatigue behaviour and a higher rutting resistance combined with a better durability – moisture resistance. Its properties conduct to a use of thia asphalt mixture on heavily trafficked roads, especially where traffic is slow and with heavy loads – truck routes, container terminals. It has excellent load-spreading properties due to its high elastic stiffness, which is mainly achieved by using low penetration grade binders, often polymer modiffied binders used. The high resistance to permanent deformation of high modulus asphalt mixtures makes it an ideal base layer for thin wearing courses in areas of high risk of rutting – combined with SMA (Stone Mastic Asphalt) as a wearing layer.
Another great advantage is from economical point of view, because this type of mixture allows thinner layers and it has a better responses to the increase of traffic loads and intensity. All these cames with savings of aggregates, maintenance and traffic interruptions.
The places where it should be used includes construction sites where a greater degree of responsibility has been passed onto the contractor, for example in the type of contract under which the contractor will design, build, finance and operate roads in return for payments related to vehicle usage.

2.4 High modulus asphalt mixtures technology problems

There are two important aspects regarding at problems raised by this technology. First, it is about the implementation of this new concept in a country with its own norms, design methods and not always comparable tests methods. Another aspect is related by the ability of local asphalt contractors to manufacture and lay the new material, including storage facilities for binder, maintenance of appropriate asphalt temperatures and appropriate compaction equipment – high modulus asphalt mixtures needs a higher compaction temperature (because of polymer modiffied bitumen) and heavy compactors for rolling. The main aspects that need to be considered during construction are:
- Ensuring that there is good bonding between subsequent layers (the application of a optimum tack coat is essential);

- Ensuring that the mixing temperature is between 160 and 180°C and that the compaction temperature never drops below 140°C;
- Ensuring that the support layer is sufficiently stiff so as to enable high modulus asphalt mixtures to be compacted to the required density;
- Ensuring that the average thicknesses are met, and particularly the minimum thicknesses;

3 Laboratory studies

Experimental study aimed that through laboratory results to highlight high modulus asphalt mixture performance when WMA technology is used. This study was carried out on two types of high modulus asphalt mixtures designed according to French Norms: an asphalt mixture – noted by "MAMR16 – HMA" and an asphalt mixture produced with "WARM MIX – L", an organic additive (WMA type) – noted by "MAMR16 – WMA". The materials (aggregates, fibber and bitumen) used to prepare the asphalt mixtures and the asphalt mixtures recipes are the following: crushed rock from Turcoaia (8/16 – 36%, 4/8 – 30%, 0/4 – 24% by mix), filler from Holcim – 10 % by mix, polymer modiffied bitumen OMV 25/55-65 STAR FALT – 4.12% by mix and organic additive (WARM MIX-L) – 0.5% by bitumen – used for WMA mix.
The organic additive used is a liquid additive mixed in bitumen, dissolving easy when bitumen is hot, without any special mixing equipment.
To highlight the performance of both new tecnologies, in Roads Laboratory of Faculty of Railways, Roads and Bridges (Technical University of Civil Engineering of Bucharest) were conducted tests for stiffness, Marshall characteristics, fatigue and permanent deformation tests. Each value is a mean value of a specified tests numbers according with european norms. In table 1 are summarized temperatures used for preparing and compacting of asphalt mixtures in laboratory. Two temperatures were used for warm mix compaction – 160°C and 145°C to highlight the influence of warm mix additive on asphalt mixture behaviour.
In figures 1 and 2, 3 are presented the results obtained for high modulus asphalt mixture using hot technolgy.

Table 1 Temperatures used at preparing and compacting of asphalt mixtures

Technology	Aggregates	Bitumen	Mixing	Compaction
	Temperature °C			
"warm mix"	150	160	160 & 145	160 & 145
"hot mix"	180	175	180	175

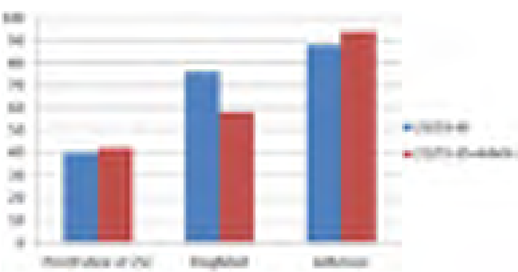

Figure 1 Changing of OMV 25/55-65 bitumen characteristics when additive is used

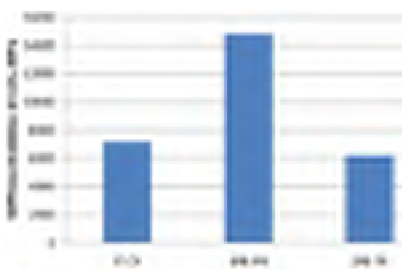

Figure 2 Asphalt mixture modulus with test type

Table 2 Physical – mechanical characteristics

Characteristic	Asphalt mixture type		
	HMA	WMA1	WMA2
	Compaction temperatures		
	175 °C	160 °C	145 °C
Density, kg/m^3	2496	2479	2465
Water sensivity, %	63	70	78
Marshall stability (S), kN	16.80	17.22	15.42
Marshall flow (I), mm	3.87	4.9	4.82
Marshall index (S/I), kN/mm	4.34	3.51	3.20

4 Pavement design depending on asphalt mix stiffness

Starting from idea that flexible pavement design based on elastic multistrat theory it must be took on consideration the values obtained in laboratory for stiffness modulus, authors propose a road structure to do case studies, according with romanian norm for pavement design (table 3).

Table 3 Road structure proposed for calculation

Road layer	Layer thickness [cm]	Stiffness modulus [MPa]	Poisson number
Asphalt mixture in wearing course	4	E_m	0.35
Asphalt mixture in binder course	5		
Asphalt mixture in base course	6		
Foundation of crushed rock	20	500	0.27
Ballast foundation	30	260	0.27
Soil type P1		100	0.27

Using the program ALIZE5 (based on Burmister theory) it was establish stress and strain state in road structure under action of standard 115 kN axle. So, it could be determined the fatigue damage ratio based on horizontal tension strain (ε_r) at the bottom of asphalt layers, and vertical strain (ε_z) at subgrade level. For calculation it was considered a 2 m.o.s. (m.o.s. means million of standard 115 kN axle) values of traffic volume Nc for a perspective period of 15 years. First case used stiffness modulus stipulated on romanian norm for pavement design (PD 177) – E_m = 3895 MPa – and the second case used the stiffness modulus obtained in laboratory – E_m = 7607 MPa. The medium stiffness was calculated using formula:

$$E_m = [S\left(E_i^{1/3} x\ h_i\right) / Sh_i]^3 \tag{1}$$

Results obtained from pavement design are presented in table 4.

Table 4 Results obtained from pavement design

Mixture stiffness	horizontal tension strain [ε_r]	vertical strain [ε_z] at subgrade level	RDO (fatigue damage ratio)	vertical admissible strain ε_z adm
Laboratory	117.6	237	0.72	272.84
Design norm	150.1	271.4	2.05	272.84

As it can see from Table 4, the structure with stiffness values from laboratory fullfil the requirements for specified traffic volum, whilst the structure with stiffness values from design norm fail the fatigue damage ratio criteria, which should be lower then 0,85.
Raising the value of traffic volume it can be find out that the structure with stiffness values from laboratory can acomodate a traffic value of 3.5 m.o.s., which correspond to a higher traffic class, according romanian norms. In this case, it can be taken in consideration lowering the thickness of asphalt layer, by taking out the binder course, without negative effects on bearing capacity of pavement structure.

5 Conclusions

Warm-mix asphalt presents an opportunity for the asphalt industry to improve its product performance, construction efficiency, and environmental stewardship. The challenge is to thoroughly research and implement this new technology in the least restrictive manner possible in order to encourage innovation and competition. Using this type of additive contribute to a slight increasing of penetraion at 25°C and a decreasing of ring and ball point, all these caming with a improvement of bitumen adhesion at aggregate, for a better durability.
Because of the additive which decrease bitumen viscosity, it can be seen a small decrease of Marshall stability for temperature of 145°C. For 160°C values are close to hot mix technology, which cand lead at conclusion that using this additive assure a longer time for asphalt mixture transportation and compaction.
The potential increase in pavement life from the combined technologies will result in significant reductions in emissions and energy costs, which is expected to outweigh the energy cost required to produce the desired additives and good quality aggregates.
High modulus asphalt mixture can assure – with a small extra cost, using high quality materials – an up to 35% reduction of asphalt layer thickness. Adding warm mix technology for this mixture, the economical and envinronmental savings are even bigger.
Although the laboratory results appear to indicate some small changes in the performance of the mixtures, they do not consider aging effects and the performance functions are based on conventional mixture data, and therefore need to be validated by field data before further conclusions can be drawn.

References

[1] Hurley, G., Prowell, B.: Evaluation of Potential Processes for Use in Warm Mix Asphalt. Alabama: National Center for Asphalt Technology, Auburn University, 2006.

[2] D'Angelo, J., Harm, E., Bartoszek, J., Baumgardner, G., Corrigan, M., Cowsert, J., Harman, T., Jamsihidi, M., Jones, W., Newcomb, D., Prowell, B., Sines, R., Yeaton, B.: "Warm-Mix Asphalt: European Practice. American Trade Initiatives". Washington, DC: U.S. Department of Transportation, 2008.

[3] Olard, F., Le Noan, C., Romier, A.: Low energy asphalt technique for minimizing impacts from asphalt plants to road work", European Roads Review, ERR No. 10, 64-75, 2007.

[4] Răcănel, C., Burlacu, A.: "Dynamic test on asphalt mixtures: considerations, performs and results", 5th Eurasphalt and Eurobitumen Congress, Istanbul, Turcia 13-15 june 2012.

[5] Răcănel, C., Burlacu, A., Moon, K.H., Mărăşteanu, M.: "Investigation of warm mix asphalt field and laboratory prepared mixture", ISAP 2012, 2nd International Symposion on Asphalt Pavements & Environment, Fortaleza, Brazilia 2012.

ROAD NETWORK MANAGEMENT IN CROATIA IN COMPARISON WITH OTHER EUROPEAN COUNTRIES

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Abstract

Road network management is a systematic process of effective maintenance, improving and exploitation of constructed roads, combining engineering principles with sound business practices and cost effectiveness. Thus, creating the conditions for more organized and flexible approach to decision-making processes in order to meet current and future customer needs is needed.

Public roads management in the Republic of Croatia is based on strategic documents, Act on Roads and planning documents. Act on Roads (Official Gazette 84/11) is the basic document governing the classification of roads, planning, construction, reconstruction, maintenance, measures to protect the traffic and roads, concessions, management, financing and supervision of the road system. Although the total road network is a unique road system due to technical, traffic and economic characteristics, there are two main subsystems in Croatia: the highways (and semi-highways) and the primary road network. Primary road network consists of state, county and local roads. Public roads management is entrusted to the County Road Administration and the City of Zagreb (for county and local roads), Croatian Roads (for state roads), Croatian highways and concession companies (Rijeka-Zagreb highway, Zagreb-Macelj highway, BinaIstra) for management of highway network. Rational and quality management of the road network is a real challenge nowadays, which must meet high traffic demands with available financial resources. Such an approach necessarily requires modern database, system for monitoring the condition of roads and facilities, the unique reference marking system of roads, preventive maintenance approach, adequate laws and regulations and the appropriate organization of road administrations and financing.

In this paper, besides the description of the road network management in the Republic of Croatia, road network classification will be described, modes of management in different European countries (such as Germany, Poland, Slovenia, Hungary, Austria, Belgium, etc.) and comparison of experiences from given countries and Croatia will be shown.

Keywords: road network, management, European countries, (road) users

1 Road network in Croatia – The legal framework

Transport sector plays an important role in Croatian economic development, with a share of around 4.7% of GDP and employee 5.2% of the working population – 58.635 employees [1]. The importance of transport infrastructure development is considered essential for economic and social development and promotion of inter-regional exchange.

The road network in the Republic of Croatia includes public roads and unclassified roads. Croatia now has in total 26.963,90 kilometers of public (categorized) roads. Decree on the Public Road Classification [2] classifies public roads as: motorways (total length 1.413,10 km), state roads (total length 6.867,70 km), county roads (9.703,40 km) and local roads (total length 8.979,70 km). All public roads are under the authority of the Minister of Maritime Affairs, Transport and Infrastructure. This method of roads classification in the state, county and local roads express the roads transport functions (clearly defined in the Decree on Standards of the Public Road Classification) [3] following the state constitution in a similar manner as in other European countries. Management, construction and maintenance of public roads in the Republic of Croatia are entrusted to several companies according to the Act on roads (Official Gazette 84/11) and concession agreements. Croatian Motorways Ltd. is a company for operation, construction and maintenance of motorways. Croatian Roads Ltd. is a company for managing, constructing and maintaining of state roads. Rijeka-Zagreb Motorway Joint Stock Company is a company for managing, constructing and maintaining of motorway Rijeka-Zagreb (part of A1, A6 and A7). These companies are 100% owned by the Republic of Croatia. Zagreb-Macelj Motorway Ltd. is a company for financing, construction, operation and maintenance of the Zagreb – Macelj motorway (A2) based on concession agreement from 2003. Bina-Istra Motorway Joint Stock Company is a company for construction, maintenance and operation of motorway Istrian Epsilon (A8 and A9) based on concession agreement since 1998.

Table 1 Key documents and planning period

Planning period	Document	Government body	Planning period
long-term	National Transport Development Strategy	Croatian Parliament	adopted in 1999
middle-term	Programme for construction and maintenance of state road network	Government of the Republic of Croatia on proposal by Ministry of Maritime affairs, Transport and Infrastructure	4 years
short-term	Construction and maintenance plan	Croatian Motorways Croatian Roads	1 year

County governments and an administrative body in the City of Zagreb are responsible for construction, operation, maintenance and protection of county and local roads. Strategy for the development of public roads, four-year programs related to construction and maintenance of roads and an annual implementation plans are key documents that define the access to road construction and maintenance (Table 1). Funds for the construction and maintenance of public roads are planned in the planning periods according to the Law on roads [4]. The basic regulation is complemented by other relevant matters – motorway designation, traffic safety, transportation in road traffic, transportation of hazardous cargo, tool tariff levels and toll collection system on motorways, calculationg the comnensation fees for the use of road land and provision of motorway services, exceptional transportation, extensive use of public roads.

2 Sources of funding

Main sources of funding for construction and maintenance activities on public roads are defined by the Law on roads [4]:
- fee of fuel, tolls paid for the use of highways and other charges related to the use of highways;
- toll to be paid for the use of highways under concession;
- charges from fuel and other charges related to the use of state roads;
- annual road charges to be paid upon registration of motor vehicles, and other fees related to the use of regional and local roads.

Since the January 1st 2014 fee charged on fuel has been redistributed: amount of from HRK 0.60 to HRK 0.20 in favor of Croatian Motorways and from HRK 0.60 to HRK 0.80 in favor of Croatian Roads expressing the investments policy in the next four-year period (Table 2).

Table 2 Plan and structure of investments according to Programs of constructing and maintaining the state road network for the period of 2009 to 2016 (in 000 HRK) [5, 6]

	Description	Motorways	State roads	County and Local roads
		plan	plan	plan
2013-2016	construction	6.896.761	5.959.880	125.680
	investment maintenance	790.125	3.194.877	1.435.299
	regular maintenance	1.034.873	1.740.000	1.955.108
2009-2012	construction	9.271.141	4.589.570	769.985
	investment maintenance	1.167.288	1.676.000	1.335.808
	regular maintenance	1.276.102	1.200.000	1.937.350

3 Road network management in the European countries

The density of the road network of high quality service in Croatia is several times higher compared with other transition countries, while in comparison with the countries of the European Union is at about at the same level.

3.1 Denmark

Denmark is a relatively small country of 42.916 km² including more than 400 islands. A well connected transport system in Denmark is therefore dependent on several types of infrastructure solutions in order to achieve a high level of mobility that integrates all regions. Total Danish road network includes motorways, main or national roads, secondary or regional roads and all other roads in a country. According to data from the Ministry of Transport [7], the Danish road network is length 73.574 km, of which 1.143 km are motorways, 379 km dual carriageways and 72.065 km all other roads. Approximately 5% of all roads are state roads while the remaining 95% are owned primarily by local municipalities. The vast majority of Danish roads and bridges are free of charge for the individual user; however, there are a few exceptions: the Great Belt Bridge joining Zealand and Funen and the Øresund Bridge linking Copenhagen to Malomö in Sweden. Several bridges connect the various islands with the Danish mainland. These bridges are essential to an integrated Danish transport system, as well as they link Scandinavia together with Continental Europe. Road Administration Structure in Denmark exists in three levels: State (1), Counties (16) and Municipalities (271). The Danish Road Directorate (Vejdirektoratet) [8] constructs, operates and maintains the state-owned road network, which comprises motorways, a number of main roads and many of the country's bridges – approximately 4.000 km.

3.2 Sweden

Sweden with area of 450.295 km² is divided into 21 counties. The length of the road network is 423.055 km and density decreases from the south to the north of Sweden [9]. Sweden has a fairly limited system of motorways. The motorways' primary purpose is connecting major cities to their surrounding areas. There are also a number of semi-motorways – roads with only 2 or 3 lanes but to which the same conditions apply as to motorways (i.e. grade-separated crossings, no slow traffic). The Swedish road numbering scheme does not distinguish between motorways and other types of roads. Sweden like, Denmark, has integrated the E-road numbers into their networks, meaning that the roads usually have no other national number. Swedish road network consist from national roads and county roads. National roads are roads of high quality and sometimes pass through several counties. Roads with lower numbers are in southern Sweden, and roads with higher numbers are in northern Sweden. The network of national roads covers all of Sweden, and has a total length of 8.769 km (not including E-roads). The national roads are public roads owned by the Government of Sweden and administered by the Swedish Transport Administration. They get a high priority for snow plowing during the winter. As of February 2008, Sweden has 59 national roads. The county roads are divided into primary, secondary and tertiary roads, of which the primary roads have the most important transport function. The Swedish Transport Administration (Trafikverket) is the Government agency responsible for the long-term planning of the transport system for road traffic and responsible for construction, operation and maintenance of the state road network and national railway network. Since 1990 the National Road Administration has been responsible for road maintenance on trunk roads and some primary county roads, including those that pass through towns. Previously it was the municipalities that were responsible for these parts of road.

3.3 Belgium

Since 1962, infrastructure construction in Belgium has been divided into three distinct zones: Flanders, Wallonia, and Brussels. Each region has different standards and construction methods when constructing roads and bridges. The road network in Belgium is made of highways, national (or regional) roads (the secondary network) and communal roads (or streets).There are also a number of orbital roads in Belgium around major cities. According data from 2010, total road network in Belgium is 154.012 km, among which there are 1.747 km of highways (both Flanders and Wallonia have approximately 900 km of highway each), 13.892 km of national (regional) roads and rest of other roads. Belgium road network has A-roads, B-roads, R-roads, N-roads, T-roads, and secondary N-roads or Provincial Routes. A-roads are motorways which connect major cities and international destinations. These are not always built as limited access facilities, and may include traffic lights and grade crossings. B-roads usually are expressway quality, and are short link routes between other points. R-roads are rings around major cities, the most famous of which being the R0 Brussels Ring. N-roads can have motorway characteristics and grade-separated interchanges but for the most part are 2 lane roads connecting secondary cities and towns. Provincial Routes are very rarely signposted, and connect smaller towns and villages [10]. The road network in Belgium is managed by regional authorities, meaning that a road section in Flanders is managed by the Flemish Government, a road section in Brussels by the Brussels government and a road section in Wallonia by the Walloon Government. Communal roads are managed at the municipal level.

3.4 France

Transportation in France relies on one of the densest networks in the world with 146 km of road and per 100 km^2, there are 1,000.960 km of roads. The French highway network consists largely of toll roads, except around large cities and in parts of the north. It is a network totaling 11.392 km of highways [11], operated by different private companies. French highway network is seventh largest highway network in the world, and, after Spain and Germany, third network in Europe. France currently counts 30.500 km of major trunk roads (routes nationales) and state-owned motorways. The county roads (routes départementales) cover a total distance of 365.000 km. The main trunk road network reflects the centralizing tradition of France: the majority of them leave the gates of Paris; trunk roads begin on the parish of Notre-Dame of Paris at Kilometer Zero. To ensure an effective road network, new roads not serving Paris were created.

3.5 Slovenia

The Republic of Slovenia a total area of 20.273 km^2, is located on the transport route linking Central Europe and the Adriatic Sea. It has a network of roads and railways that are recently renovated with the help of the European Union. The total length of the public road network of the Republic of Slovenia is more than 38.900 km. The entire road network of the Republic of Slovenia includes national roads that are owned by the Republic of Slovenia and local roads which are owned by the municipality. The total length of national roads is approximately 6.500 km. State roads include motorways, expressways, main roads (first and second rows), and regional roads (I, II and III of the order). The categorization of public roads was based on the criteria for categorization of public roads [14]. Categorization of state roads is established by the Regulation on the classification of state roads. Management, maintenance and development of the national road network – regional and main roads authorized by the Board of the Republic of Slovenia for Roads. This body conducts vocational – technical, developmental, organizational and administrative tasks for the construction, maintenance and protection of main and regional roads. Management includes several sectors. Management Division has the establishment of regional offices in major cities, while the sector headquarters in Ljubljana. Management, maintenance and development of motorways and expressways are responsible Motorway Company of the Republic of Slovenia, Inc. (DARS). DARS is a company whose founder and owner is the Republic of Slovenia. In 1995 Slovenia passed the first planning document – the National Programme of highway construction in the Republic of Slovenia, which determines the strategic, organizational and financial basis for the realization of the construction of a highway which is part of the European road network.

3.6 Poland

The Republic of Poland as a Central European country with total area of 312.679 km^2, has a well-developed network of roads, waterways, railways and air transport. As a country in the "crossroads" of Europe developing economies, Poland is becoming a modern network of transport infrastructure. Public transportation is available in most cities across the country. The national road network is adapted to the administrative division of the country, covering 412.035 km of roads and is divided into the following categories: state road, a distance of 19.182 km, the regional roads with a length of 28.423 km, the county road, a distance of 125.779 km and local roads with a length of 238.651 km. Parts of the national road network are the highways and motorways. National roads are owned by the state, while the regional, county and local roads owned by regional, county and local governments. Directorate General of state roads and highways (GDDKiA – General Dyrekcja Drog Krajowych and Autostrade) is the central government body set up to manage the national roads in Poland. In addition to managing the state roads, GDDKiA collect data and information on the public road network, road works with other governments and international organizations, is working with local governments in developing and maintaining road infrastructure. GDDKiA consists of 16 regional subsidiaries that manage the roads in their area. Headquarters GDDKiA is in Warsaw.

3.7 Germany

As a densely populated country Germany has a modern transport infrastructure. High-speed traffic has a long tradition in Germany. Germany has about 650.000 km of roads. Road network consists 12,800 km of motorways (Bundesautobahnen), 39.637 km of federal roads (Bundesstraßen), 86.474 km of state roads (Landestraßen), 91.710 km of district roads (Kreisstraßen) and 458.000 km of municipal/local roads (Gemeindestraßen). Federal roads, like motorways, are maintained by the federal agency of the Transport Ministry. In the German highway system they are ranked below motorways, but above the Landesstraßen and Kreissstraßen maintained by the federal states and the districts respectively. A Bundesstraße is often referred to as "B" followed by its number. More important routes have lower numbers. Odd numbers are usually applied to north-south oriented roads, and even numbers for east-west routes. The federal roads alone handle 30% of the total traffic load, although their share of the road network is only 2%. State roads are roads that are the responsibility of the respective federal state. They cross the boundary of a rural or urban district (Landkreis or Kreisfreie Stradt). District roads or county roads (Kreisstraßen) carry traffic between the towns and villages within a district or between two neighboring districts. District roads are usually dual-lane roads but, in a few cases, can be built as limited-access dual carriageways in densely populated areas. County roads entrusted to the counties. Local roads are the responsibility of the municipality, parish or town. District roads are usually the responsibility of the respective rural district or urban district. Local roads are roads that are the responsibility of the municipality or of the town to build and/or maintain.

3.8 Czech Republic

Road and rail infrastructure in the Czech Republic is well branched across the country, while the motorway network expands and develops. National road network of the Czech Republic under the road communications includes the following categories: highways, roads, local roads and special communications. Highways are marked with the letter "D". In the Czech Republic there are six highways. The total length of highways is 738.40 km. The oldest and most important highway that connects is the D1 Brno and Prague. According to the purpose and importance of road transport in the Czech Republic is divided into: a) Class I roads – roads designed for long-distance and international transport and those involving the so-called fast road. Fast road used for quick interstate and international transport. Marked with the letter "R", and the total length of them, is 439.10 km b) Class II roads – roads intended for transport between districts c) class III roads – roads that provide connections between individual towns or villages. Roads II and III grades are indicated by the letter "S". Local roads are used for local transportation within the municipality. Highways and roads and grades (including highways) are state-owned, while the roads were second and third grade in the property of a particular region [14]. Local roads are owned by the municipality. Highways and roads and grades managed by the roads and highways of the Czech Republic (RSD). Roads and highways is an organization established by the Ministry of Transport and Communications. RSD is currently managed with 726.90 km of motorways and express roads 391.20 km. RSD operates highways and roads and grades, cooperating with state authorities, is responsible for the maintenance of roads under its jurisdiction including facilities, participates in the development of technical regulations and road maps.

4 Conclusion

In most European countries different public organizations manage different road networks. State road authorities manage the state road network. State road network consists of corridors that connect the national administrative, economic and cultural centers of the country. Usually the state roads run through several provinces. The county road authority manages the county road network. The roads of the county road network usually connect the administrative, economic and cultural centers of the counties. It usually also connects and make use of some national roads. The local authorities are responsible for the local road network. This road network connects the administrative, economic and cultural centers in the county. Local roads usually run within the municipal borders. This method of roads classification in the state, county and local roads express the roads transport functions following the state constitution in European countries. Such strong vertical structuring is important in addressing the coordination of activities in the different layers and assures that all functions care of the widest interests of the population and the economy development.

References

[1] Ekonomski institut Zagreb: Sektorske analize, broj 25, godina 2, 2013

[2] Odluka o razvrstavanju javnih cesta (NN 66/13) Decree on the Public Road Classification (Official Gazette 44/12)

[3] Uredba o mjerilima za razvrstavanje javnih cesta (NN 34/12) Decree on Standards of the Public Road Classification (Official Gazette 34/12)

[4] Zakon o cestama (NN 84/11), Law on Roads (Official Gazette 84/11)

[5] Programs of constructing and maintaining the state road network for the period of 2009 to 2012

[6] Programs of constructing and maintaining the state road network for the period of 2013 to 2016

[7] Ministry of Transport, The Danish Transport System, http://www.trm.dk/~/media/Files/Publication/English/EUprecidency%202012/Facts%20and%20Figures-netversion.pdf, 31.01.2014.

[8] The Danish Road Directorate, http://vejdirektoratet.dk/EN/aboutus/The%20directorate/Pages/Default.aspx28.01.2014.

[9] The road network in Sweden, http://www.sna.se/webatlas/maps/text/road_network.html 29.01.2014.

[10] http://europe.aaroads.com/belgium/ 2.02.2014.

[11] List of OECD countries by road network size, http://en.wikipedia.org/wiki/List_of_OECD_countries_by_road_network_size 28.01.2014

[12] Van Rijn, J., INDEVELOPMENT: Road Network Development & Management, Edition 2005.

[13] Road Administrationshttp://www.piarc.org/en/links/road-administrations/ 2.02.2014.

[14] Stanić, A: Doprinos metodologiji ishođenja akata za gradnju, magistarski rad, Građevinski fakultet Osijek, Osijek, 2013. (mentor: Dolaček-Alduk Zlata, komentor Dimter Sanja)

LONG TERM PERFORMANCE OF ROAD MARKINGS ON RURAL ROADS: GUIDE–LINES FOR MAINTENANCE MANAGEMENT

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Abstract

A lot of data has been collected concerning road marking properties deterioration in order to develop performance prediction models for correct infrastructure management. In Italy, two types of road markings are mainly used, paint or thermoplastic, based on several components (thermoplastic binder systems, paint, epoxy, methyl-methacrylate, polyester, polyurea, waterborne paint, modified urethane). Vehicle tyres are abrasive and remove paint and thermoplastic markings, but these have different service life, due to their different surface adhesion and skid resistance properties. In the last years, the budget for transport infrastructure has been heavily cut and this has led to a decrease in monitoring and maintenance activities. Thus, road markings have been left in place longer than scheduled.
Monitoring of the long-term performance of horizontal markings began in 2008, on behalf of some Italian road authorities, with the aim of focusing on wearing course surface deterioration, carriageway perception and road marking visibility. The project initially included a limited number of sections but has been increased over time (there are now 36 sections). Collected data demonstrated that thermoplastic markings present some advantages in comparison with paint, although they are slightly more expensive. According to the results, guidelines were developed for maintenance management.

Keywords: road markings, service life, maintenance, road management, retro-reflectivity

1 Introduction

The study of the photometric characteristics of road markings is a delicate sector of research. This is because it involves the knowledge and expertise of different engineering sectors and produces results that often clash with economic and maintenance requirements. All the research developed so far, both in Italy and abroad, has recognized the importance of markings in lowering the accident rate and reducing fatalities [1][2]. An investment in the quality of road markings therefore produces economic benefits on safety and driving conditions both immediately and in the future [3].
On extraurban, non-motorway, roads a wide variety of applications and materials are used, with not always entirely satisfactory results. However, two predominate: liquid paints and thermoplastic materials, applied hot (180-200 °C). The paints, generally water-based, are low cost and have an average service life of between 6 and 18 months; thermoplastics have a higher overall cost (on average 3 or 4 times that of a paint) and a service life that, under the same stress conditions, varies between 2 and 4 years. The problem that has to be tackled is not that of just quality, which is now checked throughout the productive process, from production of the materials to the application on site, but rather that of performance. The visibility is a fundamental requirement on which savings cannot be made, because it must be constantly guaranteed over the entire route, 24 hours a day, every day of the year, and in

any environmental condition that does not preclude use of the road. If anything, the problem is how to translate the concept of visibility into measurable performances, whose values are linked both to the quality of the markings and quality of the result during the service life. A major step forward in this has been taken in recent years, since the transition began from prescriptive technical standards to performance technical standards. Today markings are defined by parameters, known as “performance characteristics”, specified in standard EN 1436 [4], to be measured directly on the applied markings with suitable instruments, at any time during the service life. These characteristics include: the Coefficient of Luminance in conditions of diffuse light, to represent daytime conditions; the Coefficient of Retro-reflected Luminance – for the nighttime; Skid Resistance – for the adhesion between tyre and road surface; the Luminance Factor of an emitting surface.
There are various studies in the literature that aim to determine the hierarchy of the factors that dominate the deterioration of the performances of road markings, with the objective of guiding the choices of inspection and maintenance times [5][6][7]. In general, they are limited by the simplification of the stresses due to traffic, climate or ageing of the materials. In these studies, simple models are extrapolated, in which no reference is made to the real effect on the performance of the transit of the individual tyre. Curves of deterioration of a general nature are therefore constructed, which depend on variables of the type: aggregate, time since application, Annual Average Daily Traffic (AADT) [5]. Taking these considerations into account and the limitations due to the lack of transparency in the transmission of information by the road management authorities [8], the authors have developed a study that directs attention on the connection between the initial performances and those in proximity to the end of the declared service life. The main aim is to demonstrate that the most delicate moment in the failure of road markings on rural roads is not so much, or only, that linked to wear and tear, but rather that of the application and the first months of service [8].

2 Methods

The performance measured in this study is the retro-reflection (R_L). This is the light reflected by the markings in the direction of the driver, when they are illuminated by the vehicle headlights. The study refers to dry markings; although the environmental characteristics at the time of measurement cannot be controlled, the moisture level of the air, by day and night, and the temperature could be monitored. The results obtained regard 36 sites, of varying lengths (for a total of 606,650 m). Each site is indicated by a geographical identifier, a work package number (W.P.), and a lot number (1-36, Fig. 1) to distinguish stretches of the same road with different markings.

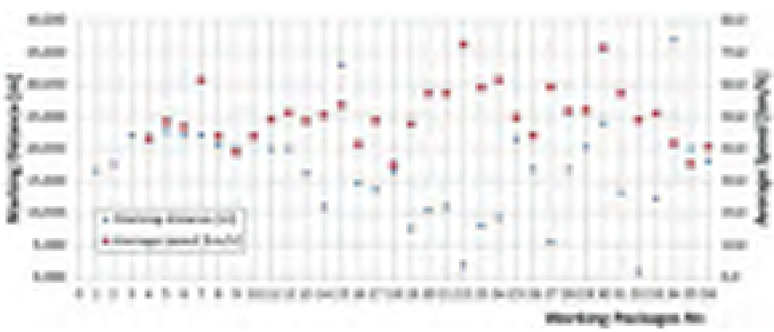

Figure 1 Work packages, speed (red) and measured markings distance (blue)

The measurements were made directly on the road with a suitably equipped and calibrated vehicle, in the presence of constant traffic in both directions; the driving speed was constrained by the conditions and therefore varied. It should be specified that most of the roads in this study are single carriageway with two lanes, one in each direction of travel (Fig. 2). The measurements of the retro-reflection of the central line were therefore interrupted when vehicles travelling in the opposite direction to that of the test were met and crossed.

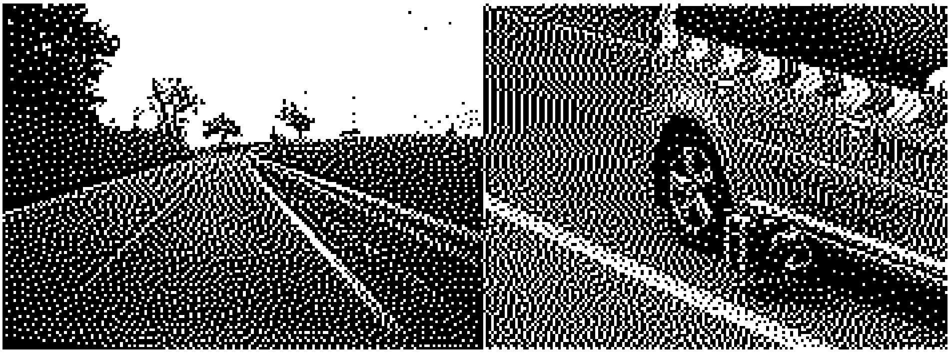

Figure 2 Road layout example and survey system

The results refer to the lines along the edge of the carriageway. All measurements were made in twice: within the first 3 weeks after the application (T=0) and within the first 6 months (T=6). During the project it was possible to follow the evolution of the damage to the pavement and markings for 3 consecutive years in 12 sites [8].
The retro-reflection characteristics were acquired with a widely-used portable instrument: the Zehntner TI® ZDR 6020 Dynamic Retro-reflectometer (ZDR, Fig. 2). The device is equipped with a rigid support for installation on a vehicle, on the right or left hand side; a measuring box, equipped with sensors for measuring the photometric characteristics; an odometer connected to a wheel, which calculates the driving speed and distance travelled; moisture and temperature sensors [9]. The rotation of the odometer is continuously controlled while travelling, thus, if the vehicle is stationary or too slow, acquisition of the R_L values is interrupted. The system measures the performances on 16 sensors; unsatisfactory values are indicated and eliminated: the points in which there is no marking and those where it is worn to the point that it offers values comparable to the road pavement. The surveyed points are spaced in order to take a value of the performance every 5 metres and an average value every 50 m, for which the GPS reference, air temperature and moisture content, average speed of the vehicle are known and a photo is taken from the vehicle dashboard.

3 Materials

The choice of investigated sites was made taking into account the available finances and prompt notification of the application: all sites for which the application date is not known have been excluded. The lack of a control on the materials used should also be taken into account, because this is not included in the inspection procedures of the markings; the paints are distinguished from the thermoplastics on the basis of a simple check of the retro-reflection values. From the results obtained and the direct surveys 14 sites can be identified where a thermoplastic product was used (5-9-12-15-19-20-21-22-23-25-26-29-34-36) and 22 sites where paint was applied.

4 Results

In agreement with previous studies [5, 10], it was decided to utilize an instrument mounted on a vehicle, because this allows measurements to be taken without the need to close the road to traffic [11, 12]. The data, both punctual and the averages extracted from the elaboration, are reported on diagrams of the performances, with the varying of the distance travelled indicated by the odometer (Fig. 3, No. 34). In these diagrams it is possible to note most of the problems linked to the survey and performance. In the example reported in Fig. 3, it can be observed that the performance measured a few days after the application (site 34) is extremely variable in both directions of travel and that it is possible to find even long stretches of unsatisfactory markings, as occurs on the right hand line between kilometres 7+450 and 9+300. This variability, which was found on the majority of investigated sites, justifies the decision to represent the performance with the statistics calculated on a 50 metres basis.

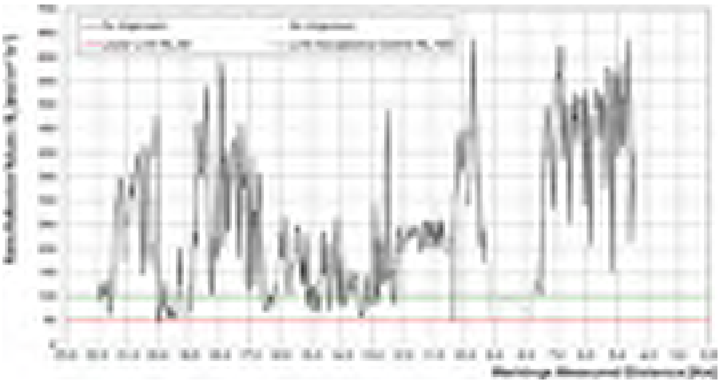

Figure 3 Retro reflection values. Stored measurements and mobile average

Fig. 4 reports the results obtained from the tests conducted in the first weeks of opening to traffic (T=0), together with the limits proposed by the classification in the standard (EN 1436). The first significant result regards the existence of values below the acceptance limits for new applications (100 mcd m^{-2} lux^{-1}). Although the average value of the performance is always above the limits (dashed line), the values are always above the minimum threshold on the entire lot only in 7 cases (nos: 1-2-4-11-12-27-28). Moreover, numerous points were observed with excellent values, above 300 mcd m^{-2} lux^{-1}, in class R5. Of these, only one was applied with paint (no: 32) and seven with thermoplastic material (nos: 12-15-19-20-21-22-23).

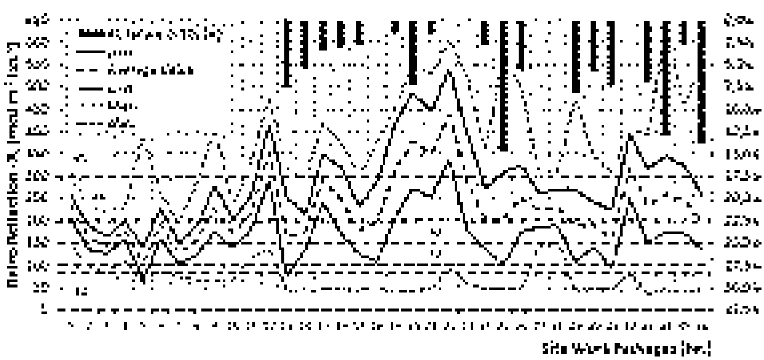

Figure 4 Retro reflection values. Statistics of all work packages (T=0)

Another important result occurs in the other sites where thermoplastic was applied (nos. 5-9-25-26-29-34-36): although there are stretches of excellent markings, in class R5, the average of the values of the initial performance is intermediate between classes R2 and R4. In site no. 5, the thermoplastic guarantees performances slightly above the threshold limit for acceptance, in class R1-2, i.e. below most of the paints (Fig. 4). The distributions of the retro-reflection values are not Gaussian and, for this reason, the indexes of dispersion, like the variance, lose statistical consistency. Despite this, if the intervals defined by the standard deviation around the average value are represented, it is possible to quantify the magnitude of the initial deficiency of the markings with greater detail. The sites with greatest problems have thermoplastic material applied (25-34 and 36): on average, more than 10% of the points surveyed are not acceptable.

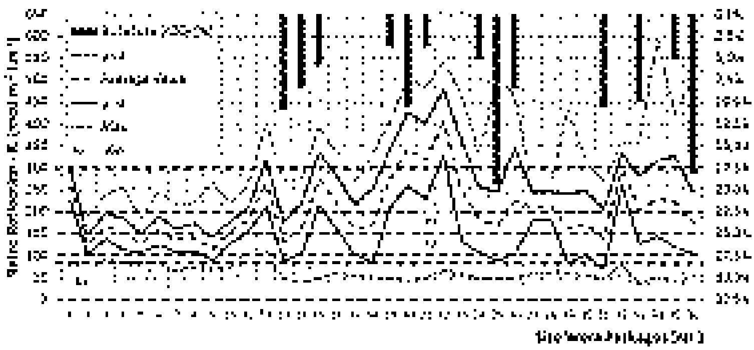

Figure 5 Retro reflection values. Statistics of performance failure (T=6)

For the surveys conducted within the first 6 months a premise should be made: the scarcity of robust and homogeneous information on traffic volumes and environmental conditions does not allow any speculation on the models of deterioration identified in the literature. The results of the elaborations demonstrate a general reduction of the performances, especially of the maximum values. Contemporarily, the number of not acceptable sections has diminished (nos. 16-17-29-30). Figure 5, while it represents the summary values of the statistics calculated on the surveys, shows that it is not simple to make a prediction that fits all the situations. There are anomalous values, both positive and negative. The distributions of the values are not symmetrical, or Gaussian, and the best results are again those offered by the thermoplastic materials. It is worth highlighting that the worst sites do not, in general, suffer an appreciable deterioration and the values of retro-reflection appear to be stable, if not actually improved (no.5). This is not surprising as the presence of walls, ditches, intersections and private accesses greatly affects the markings and cleanliness of the road surface and markings. Where unacceptable points were found after the application (T=0) on not negligible stretches, the situation has worsened (Fig. 4, points 13-14-15-19-20-21-24-25-26-31-33-35-36). Intersecting the data of the two distributions (T=0 and T=6), it appears obvious that the comparison between the two sets of values can only be made in average terms (Fig. 6). In this context, the first clear result regards the average level of the performances, which is worse than expected. The majority of thermoplastic materials applied allow medium-high performances to be obtained, in class R4 and R5, while the worst sites are nos. 5-25-29-36. The sites with the most obvious deterioration are applied with thermoplastic material (9 and 12). As regards the paints, the trend at 6 months after the application is not uniform: there are sites that improve (nos.1-3-7) and sites with deterioration of the performances (nos. 2-6-11-13-18). Despite the fact that a paint was applied in site 32 an excellent result was observed, as the values of the performance remain in class R5.

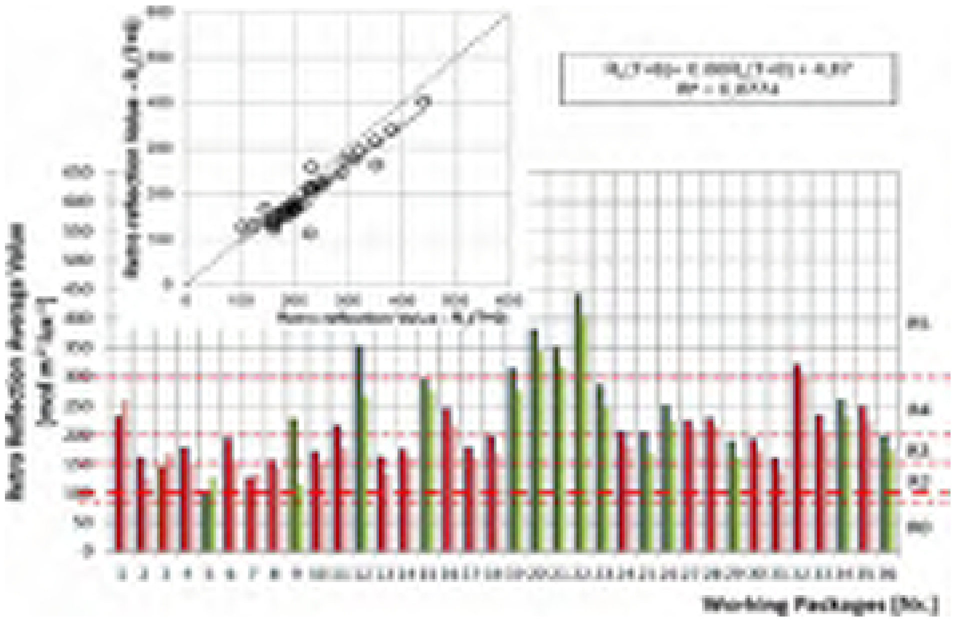

Figure 6 Retro reflection average values. Photometric performance falloff

5 Conclusions

This paper presents a detailed analysis of the performances offered by road markings, applied on roads in a prevalently extraurban context. In particular the phenomenon of deterioration of the retro-reflection has been investigated, immediately after the application and within 6 months. The sites have road markings applied with thermoplastic materials and liquid paints. The results obtained demonstrate that the modelling of deterioration curves based on the service life is not significant, because it does not take into account the numerous concomitant factors that are not considered at this level. Modelling with laws of the linear type does not allow the maintenance times adopted as standard practice to be confirmed: in the case of paints, a service life of 6-12 months is somewhat underestimated; while for the thermoplastic materials, a careful analysis is suggested at the inspection stage and the relationship does not adhere to a linear-type model. While highlighting localized anomalies, it appears clear that maintenance linked to performance characteristics should be tackled in terms of lot and with acceptance thresholds established on adequate distances. In this context it has been demonstrated that the material used, even if costly or of high quality, is not necessarily a sufficient condition to obtain high final performances and on the whole lot. While the paints offer discrete and stable performances, at least in the first months, the thermoplastic materials result as being rather delicate and variable. This suggests not only greater attention at the inspection stage but also more frequent checks, in particular where the results of the inspection fall short of the material used.

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References

[1] Al-Masaeid, H. R. and Sinha, K. C.: “Analysis of Accident Reduction Potential of Pavement Markings,” Journal of Transportation Engineering, 120 (5), pp. 723-736. 1994.

[2] Craig, W.N., Sitzabee, W.E., Hummer, J.E., and Rasdorf, W.J.: Impact of Line Location on Pavement Marking Retroreflectivity Degradation. Journal of Public Works Management and Policy, 12 (2), pp. 431-450. 2007.

[3] Abboud, N. and Bowman, B. L.: “Cost and Longevity-Based Scheduling of Paint and Thermoplastic Striping,” Transportation Research Record 1794, Paper Number 02-2170. 2002.

[4] EN 1436, Road marking materials – Road marking performance for road users.

[5] Sitzabee, W.E., Hummer J.E. and Rasdorf, W.: Pavement Marking Degradation Modeling and Analysis. ASCE Publ. 2008.

[6] Donnell, E.T:, Garvey, P.M., Sathyanarayanan, S., and Lee, D.: Method to maintain Pavement Marking Retroreflectivity. Vol1. Pennsylvania Trans. Institute. 2005.

[7] Rumar, K., Marsh II, D.K.: Lane marking in night driving: a review of past research and of the present situation, Report No. UMTRI-98-50, 1998.

[8] Pasetto, M., Barbati, S.D.: Experimental Investigation on Road Marking Distress Evolution: Beyond Testing, Quality Assurance and Maintenance Improvement, Advanced Materials Research, Vol. 723 (2013), pp. 846-853.

[9] Zehntner Testing Instruments: Instruction manual ZDR 6020 Dynamic Retroreflectometer RL, Sissach, 2009.

[10] Lundkvist, S. and Isacsson, U.: Prediction of Road Marking Performance, Journal of Transportation Engineering, pp. 133, 341-346. 2007.

[11] Migletz, J., Graham, J., Harwood, D.W., and Bauer, K.M.: Service Life of Durable Pavement Markings, T.R.R. 1749, TRB, National Research Council, Washington D.C., pp. 13-21. 2001.

[12] Goubert, L., Lundkvist, S.O.: Report of the first round robin test for mobile reflectometers. CEN/TC 226/WG2. 2011.

APPLICATION OF AN ARTIFICIAL NEURAL NETWORK IN A PAVEMENT MANAGEMENT SYSTEM

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Abstract

The era of intensive construction of new roads is behind us, and now road agencies are focused on maintaining and preserving existing pavement surfaces. As they are faced with limited funds for maintenance, it is important to best utilize resources by selecting the best maintenance strategy. Selection of an appropriate maintenance strategy is a complex task which includes factors such as current condition of the pavement, road classification, traffic volume and type of pavement distress. These factors can be automated and implemented in pavement management systems to achieve a standardised approach to road pavement assessment and management. One of the key components of pavement management systems are pavement performance prediction models, which simulate the pavement deterioration process and forecast its condition over time. One such model is the artificial neural network. This paper analyzes the possibility of using artificial neural networks in pavement management systems to evaluate existing pavement condition, and its possible application for defining the maintenance strategy of state roads. A backpropagation algorithm was applied on 481.3 km of state roads in Osijek-Baranja County, which represents 7% of total length of the national road network in Croatia. The obtained results indicated that artificial neural networks can be used for optimization of maintenance or rehabilitation strategies, and for assessment of pavement condition at the project and network level.

Keywords: artificial neural network, pavement management system, backpropagation algorithm, pavement maintenance

1 Introduction

Every pavement no matter how well designed and/or constructed will deteriorate over time [1]. Pavement deterioration is influenced by the traffic load, climate conditions, quality of construction, layer thicknesses and quality of previous maintenance and rehabilitation activities. In general, it can be said that the rate of pavement degradation increases proportionately with the intensity of its use and age. Appropriate maintenance or rehabilitation activities, if applied in timely manner, can slow down or reset degradation processes. Over the past twenty years, it has been observed that the existing, classical experience in road maintenance is insufficient, and new methods of data collection and processing are being introduced in the pavement management system. Through accelerated development of information technology and artificial intelligence, unlimited opportunities are opening for their implementation in the pavement management system. A neural network is a form of artificial intelligence that is applied in pavement management systems as pavement behaviour prediction and maintenance optimisation models. This paper analyses the possibilities of using artificial neural networks to evaluate existing pavement condition, and to define the optimal pavement maintenance strategy.

2 Artificial neural network (ANN)

A neural network is an interconnected assembly of simple processing elements, units or nodes, whose functionality is based on neuron function in living creatures. The processing ability of the network is stored in the interunit connection strengths, or weights, obtained by a process of adaptation to, or learning from, a set of training patterns [2].

2.1 Processing information in the neural network

Processing information in the neural network is carried out in units called neurons (Fig. 1). Artificial neurons have multiple entry points from which they receive information, input values, x_i. Each input value is multiplied by the assigned weight, w_i. The sum of all the weighted values, x_i*w_i represents the internal activation of the neuron, I. The transfer function f(I), applied to this sum, changes the sum of the inputs into the output value, y [3].

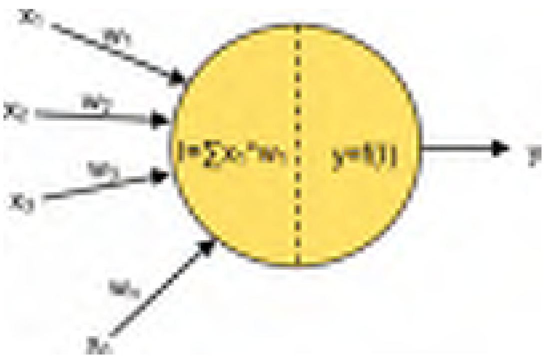

Figure 1 An artificial neuron

Neurons are connected in the network in the way that the output of each neuron represents the input into one or more other neurons. In the artificial neural network, neurons are organised in groups or layers in which the information is processed.
One of the most commonly applied neural networks, which found its place in the framework of the pavement management system, is the 'backpropagation' network. The functioning of the neural network that uses the backpropagation algorithm is described below.

2.2 Backpropagation neural network

The structure of the neural network consists of two external layers (input and output) and one or more hidden layers (Fig. 2). The network receives data by neurons in the input layer and the result of the network is given by neurons on an output layer. The hidden layers examine the interdependencies in the model and process the information of neurons, which are then forwarded to the neurons of the output layer.
The neural network is defined through two phases, the learning or training phase, and the testing phase. Prior to learning, it is necessary to define the input and output variables, and to collect data on which the backpropagation algorithm will be applied.
The backpropagation algorithm uses supervised learning, which means that we provide the algorithm with examples of the inputs and outputs we want the network to compute, and then the error (difference between actual and expected results) is calculated. The idea of the backpropagation algorithm is to reduce this error until the artificial neural network learns the training data. The training begins with random weights, and the goal is to adjust them so that the error will be minimal. In backpropagation, the scaling of local error and the increase or

decrease of weight is calculated backwards for each layer, beginning from the layer directly under the output layer, back to the first hidden layer, and the weights are then adjusted [3]. The described learning process is repeated in multiple iterations, and new weights and new errors are calculated in each iteration.

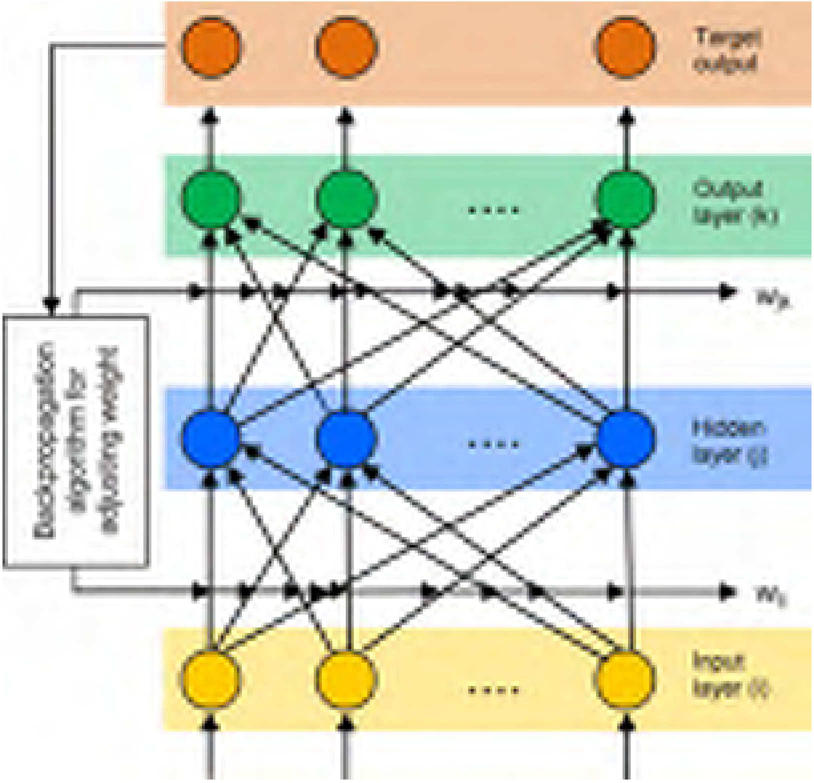

Figure 2 Structure of the backpropagation network in the learning phase

In the testing phase, the weights are fixed to the values obtained as a result of the learning phase. The network represents new input data that have not participated in the learning process. The output from the network is compared with the desired output to calculate error. Depending on the size of calculated error an assessment is given regarding the possible application of the neural network.

3 Application of ANN within PMS

As part of the pavement management system (PMS), the neural network is applicable for the purpose of assessing pavement condition, and selecting optimum pavement maintenance strategy. In this paper, pavement condition is determined with the application of the general performance indicator (GPI) as defined by the COST project 354 [4]. The maintenance strategy (MS) is defined depending on the type of required pavement rehabilitation in order to increasing pavement bearing capacity as well as driving comfort and safety [5].
The neural network input database includes data on field measurements of various types of pavement distresses (longitudinal evenness, rut depth, texture depth, surface cracks and patches). Measurements were conducted on the network of state roads in Osijek-Baranja County, over a total length of 481.3 km [5].

3.1 Preparation of the database

The road database consists of three bases, whose mutual relations are shown in Figure 3. The initial calculation database (ICD) is intended to unite the input data on the current pavement condition. The collected data are divided into segments of 1 km length, forming a database with a total of 471 samples. In this base, the mean values of input parameters were calculated, and the procedure to calculate the general performance indicator was carried out. The strategy determination database (SDD) contains the implemented initial calculation database, in such a way that the decision on the selection of individual strategies is based on the assessment of the technical parameters on the basis of the adopted criteria.

The neural network input database (NNID) consists of sets of input data (data from the initial calculation database tied to the technical parameters that describe pavement condition) and the set of output data (general performance indicator from the initial calculation database and maintenance strategy from the strategy determination database).

Figure 3 Schematic overview of databases

3.2 Learning and testing in the neural network

Neural network database is reduced by a random selection of 10% of samples that are used to evaluate the output results, i.e. a total of 421 samples were presented to the neural network. The data from the reduced neural network database were divided into two groups: a series of data in which the neural network conducts learning on 80% of the randomly selected data, while the remaining 20% form the control group on which network testing is performed. The configuration of the selected neural networks is shown in Figure 4. A neural network with backpropagation algorithm was applied in the software package NeuroShell2.0 [6].

The inputs and outputs of the neural network are connected by two hidden layers. The input layer (slab 1) contains five neurons, each of which represents one input parameter. In the first hidden layer, there are two elements (slab 2 and slab 3) with eight neurons each, while the second hidden layer contains slab 4, which also has eight neurons. The output layer (slab 5) contains two neurons, which corresponds to the number of output data. Each of the input data are connected to each of the total 24 neurons in both hidden layers, and all neurons of the output layer are connected to the final output data of the neural network.

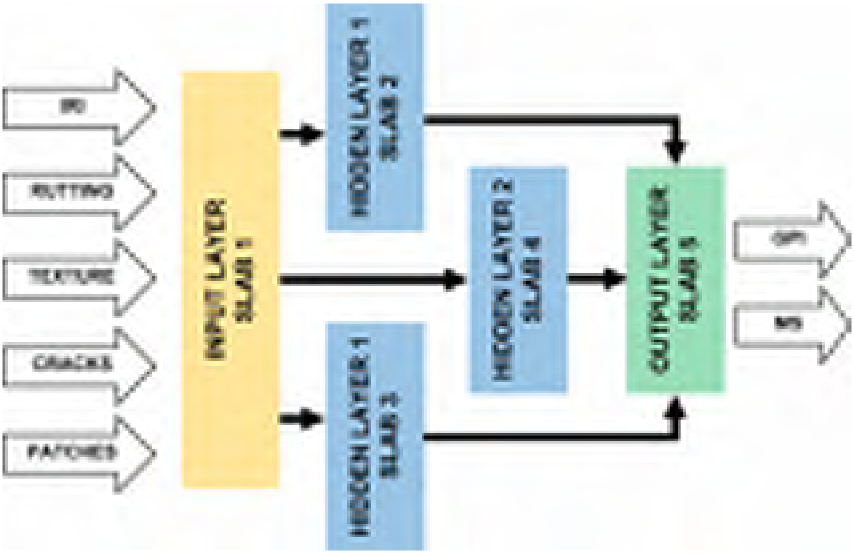

Figure 4 Schematic overview of the applied neural network

The function that results in the transfer of processed data for slabs 2 and 4 is a Gauss function, for slab 3 this is the tangent-hyperbolic function, while for the output layer this is a logical function. For all input data, a weight of 0.3 was assigned. After the learning phase, the neural network was subjected to testing. The results of statistical analysis of output data are shown in Table 1.

Table 1 Results of statistical analysis

Statistical comparison criteria	Pavement maintenance strategy (MS)	General Performance Indicator (GPI)
Coefficient of determination	0.9547	0.9678
Mean square of error	0.023	0.031
Mean absolute error	0.099	0.066
Lowest absolute error	0	0
Highest absolute error	0.900	1.759
Coefficient of correlation	0.9796	0.9851

On the basis of the obtained results, it is evident that there is a high coefficient of correlation between the results obtained in the calculation of the neural network and the known outputs.

3.3 Assessment of output results

After learning and testing of the network, a new dataset was applied (the 47 remaining samples). Only the input data, i.e. the calculated values of the technical parameters (IRI, rutting, texture, cracks and patches) were presented to the neural network. On the basis of previous 'experience', the network assessed the value of the general performance indicator and selected the management strategy. Figures 5 and 6 show the relationships between the assessed output results calculated by the neural network and the known output results.

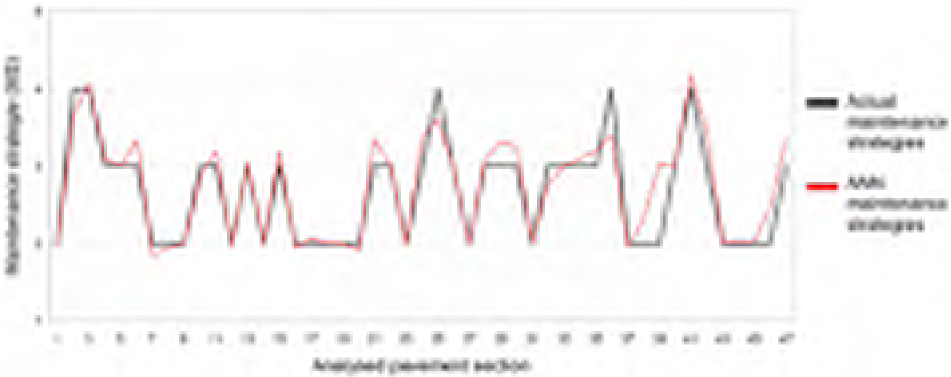

Figure 5 Selected maintenance strategies

The comparison of the actual selected maintenance strategies and strategies proposed by the neural network showed that the neural network incorrectly assessed the type of strategy on two samples, i.e. that the percentage success rate of assessment by the neural network was 95%.

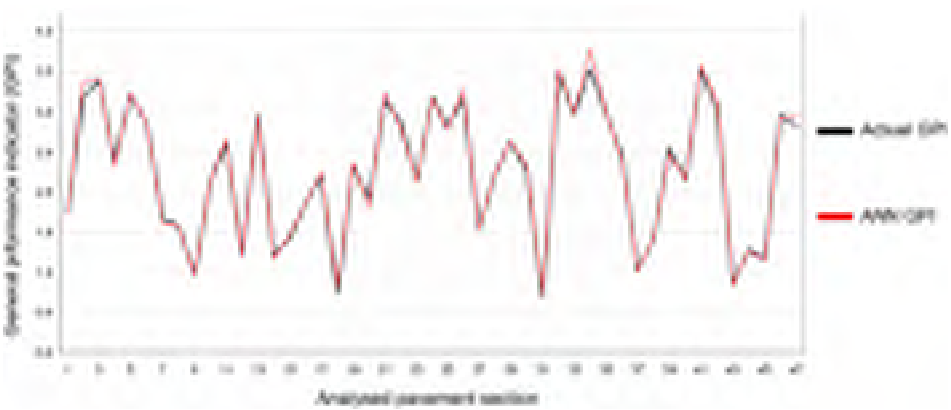

Figure 6 Calculated general performance indicator

A comparison of the assessed value of the general performance indicator with the real values ascertained that the neural network did not determine an accurate value of the GPI on six samples, and that its accuracy rate was 87%.

4 Conclusions

The primary tasks of the pavement management system are pavement distress data collection, assessment of pavement condition through an analysis of collected data, and selection of the most optimal maintenance strategy. This paper examines the possibilities of using artificial neural networks in determining the general performance indicator and selecting the appropriate pavement management strategy.
A neural network with backpropagation algorithm was selected as part of the software package NeuroShell2.0. The neural network was applied to a group of data obtained in the measurement of different pavement distresses in the state road network in Osijek-Baranja County. Statistical analysis of the output results confirmed a high coefficient of determination and correlation between the actual data and the data assessed by the neural network. In 95% of

cases, the neural network correctly determined the maintenance strategy, while the percentage of forecasting accurate values of the general performance indicator was 87%.
From the above analysis, it can be concluded that the considered neural network is appropriate for the classification of data on pavement condition for the purpose of determining the general performance indicator and optimal maintenance strategy. Its implementation in pavement management system provides a high quality tool that should facilitate decision-making in selecting maintenance procedures and rehabilitation of pavement for individual sections, sub-segments or segments of the state road network in the Republic of Croatia.

References

[1] Rukavina, T., Ožbolt, M.: Sustav gospodarenja kolnicima – prikupljanje podataka, Gospodarenje prometnom infrastrukturom, Dani prometnica 2009, Zagreb, pp. 95-175, 2009.

[2] Gurney, K.: An Introduction to Neural Networks, UCL Press, 1997.

[3] Educational Repository for Intelligent Systems; Neural Network, http://eris.foi.hr/11neuronske/nn-predavanje.html, 19.02.2014.

[4] COST Action 354 Performance Indicators for Road Pavements – Final Report, Austrian Transportation Research Association, Vienna, Austria, July 2008.

[5] Dragovan, H.: Primjena neuralnih mreža u sustavu gospodarenja kolnicima, Magistarski rad, Zagreb, 2011.

[6] NeuroShell2 Tutorial, http://www.wardsystems.com/neuroshell2.asp, 18.02. 2014.

3 ROAD TRAFFIC PLANNING AND MODELLING

THE USE OF DIFFERENT METHODOLOGIES FOR SATURATION HEADWAYS AND SATURATION FLOW RATES AT SIGNALIZED INTERSECTIONS

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Abstract

Capacity of signalized intersections relies on two basic parameters; allocated green time and saturation flow rate. Green time proportion depends on traffic demand, lane and phase configurations. Saturation flow rate is directly related to the roadway's environmental features and user's behavioural characteristics which significantly differ between locations. Although Highway Capacity Manual provides general guide for the estimation of saturation flow rate, its recommendations regarding particular values of design parameters, such as base saturation flow and capacity adjustment factors may not be universally applicable. On the other hand, there are a large number of methodologies for the field measurement of saturation headways. It is very important to conclude if the results of these methods are significantly different compared to the nature of the formatted queue. The present study was conducted to determine saturation flow rate for through traffic at a signalized junction in Thessaloniki. The analyses were based on two methodologies for the field measurements. Subsequently, the results of these methodologies compared to each other and to the estimated value of the saturation flow rate provided by the HCM with the use of suggested default values. The research's outputs are very important to draw conclusions for the adequate field methodology, depending on traffic conditions. Furthermore, it will reveal the differences between measured values of saturation flow rate and estimated values though the HCM. The outputs can be used in order to formulate a guide for the use of typical saturation flow rate values and therefore, capacity typical values applied to Greek urban traffic.

Keywords: saturation flow rate, capacity, signalized junctions, field measurements, Highway Capacity Manual

1 Introduction

The design and the operation of signalized junctions are based on their capacity, which mainly relies on the saturation flow rate in ideal conditions and a number of adjustment factors to describe prevailing geometric and traffic conditions [1]. The Highway Capacity Manual [2], Canadian Capacity Guide [3], and the Australian Road Research Board [4] provide guidelines for estimating the capacity of a signalized intersection. Although all the above manuals specify values for adjustment factors, researchers have observed significant fluctuations between these saturation flow rates and the values that derived from field measurements, because of the variations in driver's behavioural and in the roadway characteristics [1], [5].
Therefore, field measurements are thought to be the most representative method for calculating saturation flow. A great number of methodologies have been developed to measure saturation flow [6], [7], [8], [9] with headways method being the most commonly used. Saturation headways are typically estimated from the elapsed time between the 4th and the 10th

to 12th vehicles in a queue, under the assumptions (1) that there is not significant differences between short and long queues and (2) that the average headway from the first 4 to 12 vehicles is representative for long queues [10].
This study is based on measurements at a through-movement lane in a main arterial street in Thessaloniki. For the field measurements of saturation headways two methods were used. The first method (M1) includes headways between the 4th and the 12th vehicle in the standing queue, while the second one (M2) entails measurements of all the vehicles in the standing queue. M2 is the method suggested by the HCM and gives a more extensive picture of the queue discharge process [2].
The results of the two methods are being analysed to conclude if the headways derived from them are significantly different and if there is a specific method that should be used depending on the size of the queue. Subsequently, the current research is directed to the estimation of the saturation flow rate through the use of HCM's estimation method. This comparison leads to important outputs on whether the use of values suggested by HCM lead to realistic solutions when intersection operations are examined.

2 Literature review

According to Greenshields and his partners [7] the average saturation headway of all the through-moving vehicles after the 5th in the queue is 2,1 sec. Later on, in 1956, Bartle and his partners [11], following a similar process applied in the whole access, found that the average headway varied between 0,93 sec and 1,63 sec, with significant differences between different accesses. In 1970, Assmus [12] determined the average headway as the time between the 3rd and the last vehicle in the queue and in 1971, Carstens [13], proved that the 1st vehicles enters the junction 2,6 sec after the green time starts, the 2nd, 3rd and 4th vehicles have an average headway of 2,5 sec, 2,5 sec and 2,3 sec and all the following vehicles 2,3 sec. In 1976, King and Wilkinson [14] proved a decrease in the headways as the vehicle's place in queue increased and that the headway was stabilized from the 5th vehicle at 2,2 sec.
Later on, Branston and Van Zuylen [9] through linear regression resulted in the value of 1.750 pc/h/ln. In 1978, Kunzman [15], using almost the same methodology with Carstens, found out that in short queues saturation flow rate was 1.494 pc/h/through lane and in taller ones 1.726 pc/h/through lane. According to HCM 2000 and 2010 [2], the starting point for estimating saturation headway is the 5th vehicle in a standing queue. The headway is estimated by averaging the headways from the fifth to the last vehicle. Teply and Jones [16] indicated that the American, Canadian and Australian manuals have similar definition and measurement methods for saturation headway.
This is the crucial point in time when research is directed to a different approach. Typically, the discharge headway is considered to be stabilized after the 4th vehicle in the queue. Consequently, all traditional models provide the average saturation headway and flow rate. However, the uncertainty that characterises driving behaviour, prompts researchers to search for more representative tools such as micro simulation models. Generally, recent studies emphasize on the stochastic nature of discharge headways. Therefore, many models have been proposed, describing the distribution of the phenomenon. Niittymaki and Pursula [17] used both the HCM and simulation to update Finland's basic saturation flow values. They resulted in a value of 1.940 pc/h/through-lane. Jin and his partners [18] proposed a model of normal-logarithmic distribution, which depends on vehicles place in the queue. Queue length may affect saturation flow because the discharge headway may decrease in a long green phase. The fact implies that saturation headways of long queues may be lower of those estimated from the first 12 vehicles [10].
All past research proves that there is a wide range of deviation between measurements of saturation flow and suggested values of the HCM in different locations. Apart from these fluctuations, a number of studies prove that different measurement methods may lead to

different results of saturation headways. Therefore, the nature of queue and the stochastic nature of the discharge process should be deeply comprehensible in order to use the most adequate technique.

3 Methodology

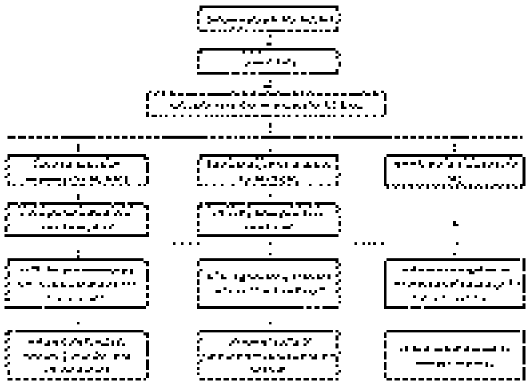

Figure 1 Research methodology

The flow of tasks for the present research is represented in detail in Figure 1. Research includes 3 basic layers; data collection and processing, formulation of the most significant concerns of the study and statistical analyses, tests and answers in the last phase of the work.

3.1 Intersection description

Data was collected in the signalized junction of Karamanli Avenue with 25th of March, where the researchers measure headways on a through lane of the approach of Karamanli Av. It is a main arterial street, where the approach has lane width equal to 3,5 m, illegal parking, almost zero grade, a bus stop into the distance of 75 m from the signal and a left-turn lane.

3.2 Data Collection and Survey Method

The data were collected with a hand held camera during morning and evening peak hours. The study was designed to be consistent with HCM data collection method. Every vehicle was considered discharged when its rear axle passed the stop line. The recording of the time points of the discharges attained through a software for processing videotapes with a chronometer with precision of 1/100 second. All the data from the videotapes were collected from one researcher to minimize biases due to different perceptions.
In every circle of the signal the measurement started when the first vehicle passed the stop line. Vehicle's discharge time was split in every 4th vehicle of the queue (e.g. the elapsed time for the 4th, 8th, 12th etc.). In this way apart from being consistent with the HCM method, the research managed to reduce measurement errors. The measurement stopped when the rear axle of the last vehicle in the queue passed the stop line.

3.3 Calculation of headways

Individual headways were calculated through the relationship below:

$$H_i = T_i - T_{i-4} / 4 \tag{1}$$

where:

i	queue position, i = 4, 8, 12…;
H_i	average headway from (i-3) to (i) vehicles;
T_i	time recorded when the rear axle of vehicle (i) passed the stopline.

The saturation headway (h) is then the average headway beyond the first four vehicles:

$$h = T_N - T_4 / N - 4 \quad (2)$$

where:

N	last vehicle in a queue, N = 8, 12...etc.

Saturation flow rate (s) is computed through the relationship:

$$s = 3600 / h \quad (3)$$

4 Saturation headways

In order to reach the objectives of the present study a number of statistical comparisons were made. Table 1 shows the number of observations for headways and the number of observations for queues length expressed with the number of vehicles in the queue for each measuring method.

Table 1 Number of observations for headways and type of queues

Headways	M1	M2	Queue	M1	M2
H4	132	132	Q≤12	132	10
H8	132	132	12‹Q≤20	-	114
H12	132	132	Q›20	-	8
H16	-	122	-	-	-
H20	-	60	-	-	-
H24	-	8	-	-	-

Firstly, the average saturation headways (h) are computed for methods M1 and M2, regardless of the queue size. The value of M3 is estimated from HCM 2000. Subsequently, saturation flow rates (s) were derived through Equation 3. Two-way t-test between the h estimations for M1 and M2, regardless the type of queue, showed that they are not significantly different in a significance level of 95%. On the other hand, the estimated value of the method M3 differs significantly from field measurement methods (Table 2).

Table 2 Basic data analysis

Method	h [s]	Std. dev. [s]	Observations	S [pc/h/ln]
M1	2,0794	0,20676	132	1731
M2	2,0895	0,20588	132	1722
M3	2.3245	-	-	1549

The results of the application of methods M1 and M2, depending on queue's size are presented in Table 3. In both methods long queues present significant differences in headways than short ones. Furthermore, the differences between M1 and M2 increase as the length of queue increases. Contrary to Bonneson [19] and Li & Prevedouros [10], study queue size appears to be a strong measure of pressure to the headways, a fact that is directly correlated to Greek driver's behaviour.

Table 3 Basic data analysis

Queue	M1's h[s]	M2's [s]
short	2,095	2,095
medium	2,078	2,091
long	2,075	1,941

The means and standard deviations of headways in method M2 are displayed in Figure 2 which shows that the minimum headway is reached between the 5^{th} and the 8^{th} vehicle, as the HCM implies. The difference from HCM is that from that point on headways don't obtain a stable value. Figure 3 proves that, contrary to the HCM, after the 7^{th} vehicle the fluctuations of headways are large and become larger after the 20^{th} vehicle. Therefore, headways don't reach a stable value and the diagram cannot be smoothed to fit in HCM. In long queues driving behaviour seems to be so unpredictable that headways cannot be simulated according HCM suggestions. The similarities in driving behaviour between Greek and American drivers are obvious.

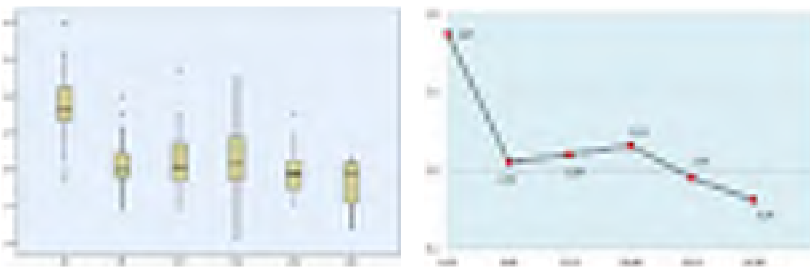

Figure 2 Means and std. dev. of headways in method M2

Figure 3 Analytical diagram of the saturation headways

According to Figure 4, the saturation flow rate obtains a first peak between the 5^{th} and the 12^{th} vehicle in the queue. Afterwards, its value reduces progressively and obtains its minimum value between the 12^{th} and 16^{th} vehicle. After this point, it starts increasing once again.

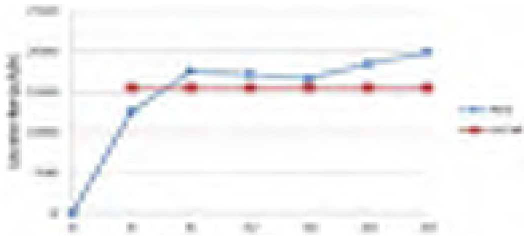

Figure 4 Saturation flow rate in method M2 and M3 (HCM)

5 Headway elongation/compression

The investigation of the potential elongation or compression derives through the examination of headways after the 12th vehicle. The results of t-tests show that the observed last headway is significantly different from the preceding average when the last headway refers to the 16th vehicle of the queue and also to the 24th one. In the first case the headway appears to be much higher than the average headway of the previous vehicles and in the second one the headway is smaller than that of the preceding vehicles. The traffic analysis of this approach is inconsistent with the HCM, in which it is essentially assumed that the saturation headway remains stable until the end of a standing queue. All these elements lead to the conclusions that when the standing vehicles are 13 to 16 drivers don't feel so much pressure to pass through the junction resulting in elongated headways. On the other hand, the last four drivers of a long queue (>20 vehicles) compress their headways to take advantage of the expiring green.

6 Conclusions

The results of the present study derived from a detailed analysis of headways on short, medium and long queues with heavy random arrivals at a busy approach of one of Thessaloniki's main central arterial roads and are outlined below.

- Saturation flow rate values deriving from HCM are significantly lower from the saturation flow rate calculated from field measurements, by methods M1 and M2 The differences in driving behaviour and in roadway's environment between Greece and USA are undeniable.
- Contrary to the HCM, even though saturation headway reaches its minimum value between the fifth and 12th vehicles, it does not obtain a stable value from that point on.
- In long queues both methods M1 and M2 provide smaller values of saturation headways than in short queues. Queue size appears to be a strong measure of pressure to the headways, a fact that is directly correlated to Greek driver's behaviour.
- Long queues present significantly different results with the use of the two methods M1 and M2. Further research must be carried out to conclude which of these methods is the most appropriate.
- In short queues drivers don't feel so much pressure to pass through the junction resulting in elongated headways. On the other hand, in long queues drivers compress their headways to take advantage of the expiring green.

All these conclusions lead the present research to suggest field measurements for saturation headways in Greece and additionally, different treatment between short and long queues through different methodologies of measurements. Future endeavours on this subject include the enrichment of the databases with data both in measurements in the same junction and in other intersection approaches, which will reduce potential local effects from this single approach analysis.

References

[1] Alam, J.B., Osra, K.A., Al-Bar, H.O. & Zahran, S. Z.: Signalized Intersection Capacity Adjustment Factors for Makkah, Saudi Arabia, Canadian Journal of Transportation, Volume 4, Part 1, pp. 55-66, 2011.

[2] Transportation Research Board of the National Academies: Highway Capacity Manual, Washington D.C., 2010.

[3] Teply, S.: Canadian Capacity Guide for Signalized Intersections, Institute of Transportation Engineering, Vancouver, B.C., 1995.

[4] Akcelik, R.: Traffic Signals: Capacity and Timing Analysis, Research Report ARR 123, Australian Road Research Board, Vermont, South Victoria.

[5] Mukwaya, R. & Mwesige, G.: Saturation Flow Rate for Through-Traffic at Signalized Junctions in Kamplala, 2th International Conference on Advances in Engineering and Technology, India, pp. 464-470, 2012.

[6] Transport and Road Research Laboratory: A Method of Measuring Saturation Flow at Traffic Signals, Road Note No. 34, London, 1963.

[7] Greenshields, B.D., Shapiro, D. & Ericksen, E.L.: Traffic Performance at Urban Intersections, Bureau of Highway Traffic, Technical Report No. 1, Yale University, 1947.

[8] Leong, H.J.W.: Some Aspects of Urban Intersection Capacity, Australian Road Research Board, pp. 305-338, 1964.

[9] Branston, D. & Van Zuylen, H.: The Estimation of Saturation Flow, Effective Green Time and Passenger Car Equivalents at Traffic Signals by Multiple Linear Regression, Transportation Research, Vol. 12, pp. 47-53, Pergamon Press, Great Britain,1978.

[10] Li, H. & Prevedouros, P.D.: Detailed Observations of Saturation Headways and Start-Up Lost Times, Transportation Research Record 1802, pp. 44-53, 2002.

[11] Bartle, R.M., Skoro, V. & Gerlough, D.L.: Starting Delay and Time Spacing of Vehicles Entering Signalized Intersection, Highway Research Board Bulletin 111, pp. 33-41, Washington D.C., 1956.

[12] Assmus, W.E.: Operational Performance of Exclusive Double Left-Turn Lanes, Evanston, 1970.

[13] Carstens, R.L.: Some Traffic Parameters at Signalized Intersections, Traffic Engineering 41, pp. 33-36, 1971.

[14] King, G.F. & Wilkinson, M.: Relationship of Signal Design to Discharge Headway, Approach Capacity and Delay, Transportation Research Record 615, pp. 37-44, 1976.

[15] Kunzman, W.: Another Look at Signalized Intersection Capacity, ITE Journal 48, pp. 12-15, 1978.

[16] Teply, S. & Tones, A.M.: Saturation Flow: Do we Speak the Same Language?, Transportation Research Record 1320, TRB, pp. 144-153, Washington D.C., 1991.

[17] Niittymaki, J. & Pursula, M.: Saturation Flows at Signal-Group-Controlled Traffic Signals, Transportation Research Record 1572, pp. 24-32, TRB, Washington D.C, 1997.

[18] Jin, X., Zhang, Y. & Wang, F.: Departure headways at signalized intersection: a log-normal distribution model approach, Transportation Research Part C, pp. 318-327, 2009.

[19] Bonneson, J.A.: Modeling Queued Driver Behaviour at Signalized Junctions. Transportation Research Record 1365, pp. 99-107, TRB, Washington D.C., 1992.

COMPARATIVE STUDIES REGARDING TRAFFIC FLOW IMPROVEMENT SCENARIOS USING SOFTWARE MODELLING AND REAL MEASURED DATA

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Abstract

The purpose of this paper is to study and evaluate the effects of different road network changes on traffic conditions, based on measured stream characteristics and computer-based modelling. The studied perimeter is situated at the Western edge of the city of Cluj Napoca, Romania, on the borderline between urban and suburban areas. At the moment, the European E60 road is the only functional road link which connects the city to the Western suburb of Florești. Population increase and suburban expansion led to high traffic volumes, especially during the morning and afternoon commuting peak hours. As traffic is expected to further increase in the future, we conducted this comparative study to see which solution would have the best impact on traffic conditions in the studied area.
The traffic data we used was collected by a high-speed weigh-in-motion system, installed on the studied E60 road link. We used this real measured data to make a 15-years traffic forecast, based on Romanian regulations. After observing that road capacity has already started to be exceeded during peak hours, we studied two possible traffic flow improvement scenarios. In order to obtain a better look on the impact of the proposed measures, we studied a broader area, including parts of the adjacent neighbourhoods. The computer-based transport modelling and traffic assignment was done using Citilabs' Cube Voyager software. After setting up the road network and modelling the existing situation, we evaluated the impact of each scenario on the traffic volumes, route assignments and travel times. Results show the differences between the existing traffic conditions and the proposed possible future scenarios, which could be adopted to improve the level of service of the streets in the studied area. In the end, we reached a conclusion regarding the recommended solution.

Keywords: peak hours, weigh-in-motion, transport modelling, volume, travel time

1 Introduction

The development and evolution of human settlements have always been closely linked to the existence of a line of communication between them. As technology and transportation evolved, especially in the latest centuries, we have witnessed a rapidly changing and developing world [1]. Growth management and traffic congestion are relatively new, but difficult issues [2]. In the situation we approached, these are actually the main problems which this study aims to provide a solution for. The studied area is located in Romania, at the Western edge of the city of Cluj Napoca. The expansion and the rapidly increasing population of the Florești suburban area should have been backed up by sustainable urban development plans, traffic engineering measures and an efficient policy regarding public transportation [3]. The lack of these measures led to traffic congestion and increasing travel times. As traffic volume is expected to increase during the next years, we considered two possible solutions to improve traffic stream characteristics.

2 Traffic data

2.1 Location and traffic monitoring system

The studied perimeter, shown in Fig. 1, is situated at the Western edge of the city of Cluj Napoca, Romania. This city is a major cultural, industrial, academic and business centre in Romania. According to the 2011 population and housing census [4], around 325,000 people live in the city. The European E60 road (National road DN1) is the main road which connects it to the Hungarian border to the West, as well as to Central and Southern Romania. In the analysed section, the road has four lanes and an East-West layout.

Figure 1 Study location

Approximately 4 km West of Cluj Napoca lies the suburban city of Florești. According to the 2011 census [4] and to Toșa et al. [3], the population of this settlement has known an increase of about 260% since 2007, from 8,600 to almost 23,000 inhabitants. This spectacular evolution led to the constant increase of traffic volumes on the only functional road link between the two cities, especially during the weekdays morning and afternoon commuting peak hours. Since April 2013, a high speed weigh-in-motion (WIM) system installed on the E60 road link (Fig. 1) has been functional. The system we installed has a piezo-loop-piezo configuration. Not only does it count and classify the passing vehicles, but it also uses piezo-electric sensors to weigh them [5].

2.2 Traffic stream parameters

All collected data was classified according to the direction of travel: Eastbound and Westbound. We extracted the traffic volumes from the WIM data and divided them into five vehicle categories, based on the Romanian standard for vehicles equivalation [6] and our knowledge of local traffic characteristics. We converted each of the five vehicle categories we considered into Passenger Car Equivalents (PCE), using the conversion factors provided by the Romanian standards for vehicles equivalation [6] (Table 1) and highway capacity evaluation [7]. As we also want to study traffic evolution over the next 15 years, we extracted the evolution coefficients provided by Romanian norms [8] (Table 1).
The current and estimated average weekday traffic [2] we obtained from the WIM data for the E60 link that we studied are shown in Fig. 2. We selected the 7.00–9.00 hrs and 17.00–19.00 hrs as peak intervals for traffic heading East and West, respectively.

Table 1 Vehicles data

#	Vehicles	PCE conversion factor	Evolution coeff. (2014 – 2029)
1	motorcycles	0.50	0.64
2	automobiles	1.00	2.22
3	light commercial vehicles	1.20	2.10
4	other light vehicles	3.00	1.75
5	large goods vehicles (LGV), buses, coaches	3.50	1.95

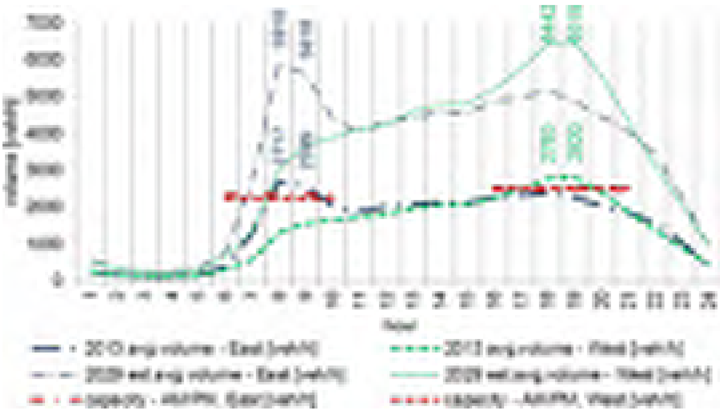

Figure 2 Average weekday traffic

3 Traffic assignment

The modelling and analysis of the road transport system that we studied was done using Citilabs' Cube Voyager software. The computer–based transportation forecasting we carried out was based on the following main steps:
- setting up the existing road network;
- establishing the position of the origin and destination zones and connecting them to the network;
- building the trip matrices for the morning and afternoon peak hours;
- building the traffic assignment models for the existing situation;
- adjusting the model inputs so the results match the WIM measured average traffic volumes;
- setting up network improvement scenarios and forecasting their impact on the traffic volumes and route assignments.

3.1 Network model, path building parameters and existing situation

The first step of the traffic assignment process was generating the Cube Voyager network file, shown in Fig. 3. We focused on building a traffic simulation model with macroscopic characteristics, modelling only the main road links and considering five origin–destination zones (1-4, 11) to the West and six (5-10) to the East. The main link between the two areas is represented by the E60 road, where the WIM system is installed. The road network model was also detailed with turn penalties and signalised intersections data, including traffic signals phases, cycle times and delays.

Figure 3 Model of the existing road network

We chose time as the cost of travelling on a certain path between two zones. We prepared the morning and afternoon trip matrices using data provided by traffic surveillance cameras, knowledge of local travel characteristics, data provided by Toșa et al. [3] and WIM data. One of the most popular optimisation algorithms to obtain network equilibrium and minimum–cost traffic paths, based on the Wardrop equilibrium principles [9], is the iterative Frank–Wolfe algorithm [10]. The general volume–delay function used to evaluate the travel time on a link is expressed by Eq. (1):

$$t(V) = t_0 \cdot \left[1 + 0.15 \cdot \left(\frac{V}{C}\right)^{\alpha}\right] \tag{1}$$

where:

$t(V)$	average link travel time;
t_0	free–flow link travel time;
V	traffic volume;
C	link capacity;
α	exponent.

Although the formula expressed by Eq. (1) is widely used, according to Spiess [11] it has a few disadvantages, especially in the case of roads with higher capacity, when α increases. Therefore, we chose to use the conical function proposed by Spiess [11] and described by Eq. (2):

$$t(V) = t_0 \cdot \left[\frac{2\alpha - 3}{2\alpha - 2} - \alpha \cdot \left(1 - \frac{V}{C}\right) + \sqrt{\alpha^2 \cdot \left(1 - \frac{V}{C}\right)^2 + \left(\frac{2\alpha - 1}{2\alpha - 2}\right)^2}\right] \tag{2}$$

with the same symbols as in Eq. (1). We adopted α=8 for four-lane roads and α=4 for one and two-lane roads.

3.2 Scenarios

After building the traffic assignment models and adjusting the model inputs so that the results match the WIM measured average traffic volumes, we obtained the current situation. Considering this situation and the evolution of traffic over the next 15 years, we carried out a traffic forecast using the Cube Voyager application, on two improvement scenarios. The first scenario we considered for the forecast, shown in Fig. 4, was the modernisation of an existing dirt road to the North of the E60 and the opening of a new road link, to the South. At the same time, we considered replacing a roundabout and a T intersection currently used in Florești with two signalised intersections. The second improvement scenario, shown in Fig. 5, involves reversible lanes on the main East-West link, connecting zones #8 and #11, in addition to the changes considered in the first scenario. During the morning peak hours, we considered three lanes accommodating traffic heading East and one lane for the opposite direction, and vice versa for the afternoon peak hours. For the three-lane roads, we considered α=10 in Eq. (2).

Figure 4 Scenario #1

Figure 5 Scenario #2

3.3 Results

We carried out the route assignment on the existing road network and on the two scenarios described, using traffic data collected in 2013 and estimated for the year 2029. A synthesis of the results we obtained is shown in Table 2, Table 3, Fig. 6 and Fig. 7.

Table 2 Peak hours average traffic volumes on E60 main road link – 2014

traffic heading	2014					
	AM (morning)	PM (afternoon)				
	existing	scen.#1	scen.#2	existing	scen.#1	scen.#2
	vol.[PCE/h]	change [%]	vol.[PCE/h]	change [%]		
E	2658	-27.8	-24.5	2332	-36.7	-45.2
W	1441	-40.2	-48.9	2801	-29.5	-16.4

Table 3 Peak hours average traffic volumes on E60 main road link – 2029

traffic heading	2029					
	AM (morning)	PM (afternoon)				
	existing	scen.#1	scen.#2	existing	scen.#1	scen.#2
	vol.[PCE/h]	change [%]	vol.[PCE/h]	change [%]		
E	5764	-18.8	-14.8	5038	-8.6	-10.8
W	3362	-28.3	-44.6	6480	-31.9	-23.9

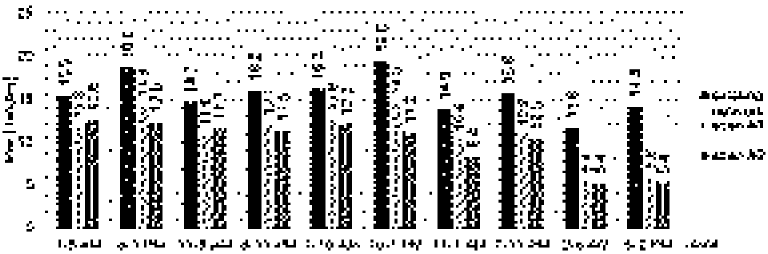

Figure 6 Average travel times – busy routes, 2014 traffic volumes

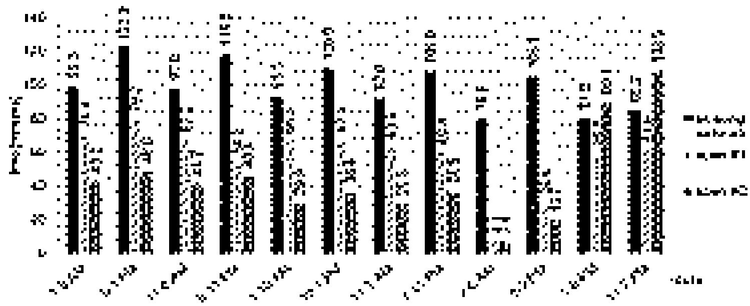

Figure 7 Average travel times – busy routes, 2029 estimated traffic volumes

4 Conclusions

As we can see, maintaining the E60 road as the only road link between the zones we studied will lead to forced traffic flow and traffic congestion. If the evolution of traffic follows the pattern we estimated, in 15 years' time the travel times on the main routes we studied will be, on average, 6.6 times longer than now.

The two proposed scenarios would have a similar effect on the busy routes average travel times, reducing them with an average of 31.5% and 35.8%, respectively (Fig. 6). In the short term, we estimate that scenario #1 would have a better impact than scenario #2, reducing the traffic volume on the main link with approximately 30-40% (Table 2). However, in the long term, even though the peak hours average traffic volumes on the main link would be similarly reduced (Table 3), scenario #2 implies significantly shorter travel times. In 15 years' time, we estimate that the current travel times between main zones of interest would be reduced with 45.4% in scenario #1 and 67.6% in scenario #2 (Fig. 7). On the other hand, as shown in the last two clustered columns of Fig. 7, adopting reversible lanes has a negative impact on the travel times on some of the routes.

In conclusion, we estimate that scenario #1 would have a positive impact on traffic flow in the studied area, especially in the short term. The two alternate routes and the revised traffic signalisation would considerably reduce traffic congestion and average travel times. Reversible lanes would be a suitable solution in the long term, but adopting them should be reconsidered based on the traffic evolution up until that point. However, any route network improvement scenario should be sustained by further measures such as introducing an efficient public transport system and implementing a sustainable urban mobility plan.

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References

[1] Rodrigue, J.P., Comtois, C. & Slack, B.: The Geography of Transport Systems, Routledge/Taylor & Francis, New York, United States, pp. 171-203, 2006.

[2] Roess, R.P., Prassas, E.S. & McShane, W.R.: Traffic Engineering, Third Edition, Pearson Education Inc., Upper Saddle River, New Jersey 07458, 2004.

[3] Toşa, C., Antov, D., Köllő, G., Rouk, H. & Rannala, M.: A Methodology for Modelling Traffic Related Emissions in Suburban Areas, Transport, Taylor & Francis, United Kingdom, http://dx.doi.org/10.3846/16484142.2013.819034, 2013.

[4] www.recensamantromania.ro, 2011.

[5] HI-TRAC® 100 High Speed Weigh-in-Motion Data Collection & Pre-selection System. Utilising Hi-Comm 100 Software – System Overview, TDC Systems Limited, Weston-super-Mare, North Somerset, England, United Kingdom.

[6] SR 7348-2001: Road Works. Vehicles Equivalent for Traffic Capacity Determination, Romanian Standard, 2001.

[7] PD 189-2000: Norm for the determination of public roads capacity, Romanian Standard, 2000.

[8] AND 584-2012: Norm for the evaluation of the computing traffic for the structural and capacity design of roads, Romanian Standard, 2012.

[9] Wardrop, J.G.: Some Theoretical Aspects of Road Traffic Research, ICE Proceedings: Engineering Divisions, 1(3), pp. 325-362, doi: 10.1680/ipeds.1952.11259, 1952.

[10] Frank, M. & Wolfe, P.: An Algorithm for Quadratic Programming, Naval Research Logistics, 3, pp. 95-110, doi:10.1002/nav.3800030109, 1956.

[11] Spiess, H.: Technical Note – Conical Volume-Delay Functions, Transportation Science, 24(2), pp. 153-158, http://dx.doi.org/10.1287/trsc.24.2.153, 1990.

TRANSPORT DEMAND MODELING FOR NATIONAL PARK MAVROVO

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Abstract

National parks are protected areas which are real representative of natural sourses and eco systems, and require special care in part of transport demand modeling. In this paper the four steps model with features of region to get the modal values, traffic assignment and forcast for 10 years, using the powerful software PTV Vision VISUM on particular case, for the National park Mavrovo will be shown.

Keywords: National park Mavrovo, model, forecast

1 Introduction

Traffic planning, existence and conetivity of quality road network is main factor for development on each teritory. In this paper, calculation of the modal values and traffic assignment is made for national park Mavrovo. Specific for regional planning is that settlments are represented as nodes.
Field data are colected for making the fourstep model in PTV VISUM software. To perceive the need of improvement and restoration of the traffic and touristic infrastructure in the park, it is made a forecast of the traffic demand for next 10 years in PTV VISUM software. At the end, it is made a comparison of the modal values for verification of the output results, and also to identify the difference between actual and forecasted condition.

2 Data collection for traffic planning in National park Mavrovo

For creating a quality planning of traffic demand, collection of reliable data is needed to obtain correct and real output.

2.1 Statistical method

For the necessities of the paper, it is made collection of data for number of citizens for each settlement, job places by activity (education, agriculture and ranching, administration, catering) and intensity of traveling to and from National Park Mavrovo.

2.2 Survey method

The survey was made of the whole region of National park Mavrovo on 06.04.2013, with purpose to collect data for job places and attendance of tourist objects. Processed data are used as input in the model.

3 Transport demand modeling

Modeling is made for actual traffic condition in National Park Mavrovo, with use of the four-step model, traffic assignment and forecast in PTV Vision VISUM software. On Figure 1 with green color is shown the traffic network for researched territory. Each link has it's own capacity and speed which correspond with the actual situation.

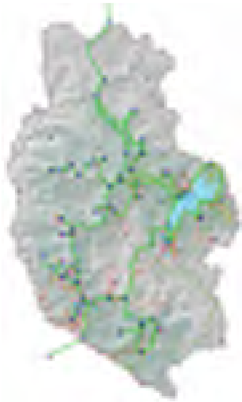

Figure 1 Graphic display of road network

3.1 Zoning

Urban zoning is a part of spatial and urban planning, and is a division of the territory covered by the plan coverage of different functional zones with different purposes. Zoning is the process of selecting areas with specific purpose and labeling (settlements, recreational areas, education, administration, tourism and catering). Zones, according of the characteristic purpose, are origin and destination of trips according the movement needs of the population. The territory of National park Mavrovo is divided on 12 traffic zones and 3 external zones, which are separated with roads, natural borders, pedestrian and bike paths.

Figure 2 Separation of the National Park Mavrovo by zones

3.2 Model choice for transport demand

Model choice depends from the input which is used. In this case, the fourstep model is selected with characteristics of generation, distribution, allocation and trip assignment. For internal calculation in the PTV VISUM sotfware, this model uses the values of zone atributes (number of residents, job places in education, administration, tourism, catering, agriculture, and trips between and inside the zones). Figure 3 shows the selection of the fourstep model in the software.

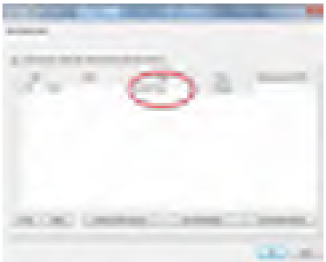

Figure 3 Selection of four step model

4 Traffic demand for National park Mavrovo

Modal values represents trip assignment on certain streets, which actually is trip distribution on transport network. For calculation of the modal values with the fourstep model, it is neccesary activation of all generations, distributions, and assignment, while the Skim matrix should be deactivated. Intensity of trips on roads is shown on Figure 4 with red color.

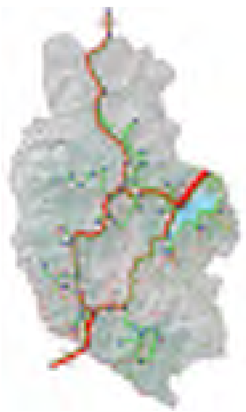

Figure 4 Traffic demand of National Park Mavrovo

5 Traffic demand forecast of NP Mavrovo

Traffic demand forecast actually is prediction of change on movement needs for certain time period. The process of forecast for next 10 years, is made with method of estimation of population increase, which linearly increases the mobility. Modal values for transport demand in the next 10 years is shown on Figure 5.

Figure 5 Transport demand forecast

6 Verification of output modal values

Verification is independent procedure which is used to confirm to what extent the model outputs match the results obtained from the field. To verify how modal values from PTV VISUM match with the real traffic flow on certain section, comparative analysis is made. According to Fund for regional and national roads, on the section Boshkov most – Debar, average by day are passing around 650 vehicles in both directions. Respectively, on this section, the software calculates that daily are passing 654 vehicles in both directions.

Figure 6 Modal values on the section Boshkov most

The dispersion between calculated and real value is less than 1 %, which is affirmation of the hypothesis that output results are valid.

7 Comparison of current and forecasted modal values

Comparative analysis of output results is especially important to see if the forecasted values are real and match the growing movement needs in the future. Comparison is made for the current and forecasted modal values for the next 10 years, calculated in the PTV Visum, on a section of the entrance to the National Park Mavrovo road in direction of cities Kicevo and Gostivar. On Figure 7 is presented the section with current transport demand, which is 945 trips (827 for entry and 118 for exit).

Figure 7 Current modal values

On Figure 8 is shown the forecasted transport demand for the next 10 years, and it's value is 1170 trips (1025 – entry, 145 – exit).

Figure 8 Forecasted modal values

According to calculated values with the software, the forecasted values for transport demand are for 24 % greater than current.

Conclusion

Unlike analytical approaches for calculating the transport demand and forecast , PTV Vision VISUM software provides fast and accurate calculation and offers a graphical representation of the output results. Verification of current and comparison of the forecasted modal values, are part confirming the high reliability of the outcomes of PTV Vision VISUM. Calculation of current and forecasted increase of future transport demand is crucial for understanding the mobilities in National Park Mavrovo, and is the basis for taking steps in the engineering maintenance and improvement of transport infrastructure .

References

[1] Atanasova, V.: "Traffic Planning", internal lectures, University "Ss. Kliment Ohridski", Faculty of Technical Sciences, Bitola, 2010.

[2] PTV America, VISUM User Manual, Version 7.50. 2. MODEL DESCRIPTION. 2.1. Network Model. 2-3. 2.1.1. Transport systems, April 2001.

[3] Atanasova, V., Detelbah, S., Markovik, L.: PTV Vision VISUM Manual, Faculty of Technical Sciences, Bitola, 2012.

[4] Hadjipetkova, K., Ilievski, D.: Seminar paper "Sustainable development of transport infrastructure for spatial plan of the National Park Mavrovo" – Faculty of Technical Sciences, Bitola, 2013.

[5] www.npmavrovo.com.mk, 15.03.2013

[6] www.stat.gov.mk, 19.03.2013

IMPACTS OF THE CONSTRUCTION OF THE PLANNED RESIDENTIAL AND BUSINESS COMPLEX ON THE ROAD NETWORK OF THE CITY OF MOSTAR

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Summary

Mostar municipality is an area of great significance for Bosnia and Herzegovina, both for the natural resources and cultural heritage as well as for the fact that it occupies the central part of the territory of Herzegovina and extends to the middle part of the Neretva River. Therefore, it has become an important traffic point over which cross the roads of European, national and regional importance. This paper presents the results of the research of traffic on the road network of the City of Mostar over the plan period. Within the framework of the research, the analyses were performed on the impact of the construction of planned roads in relation to the current traffic volume on the existing ones. Also, the impact of the construction of the planned residential and business area on the traffic volume was explored and estimated.

Keywords: planning, traffic, road network, traffic volume

1 Introduction

Mostar is situated on the Pan-European transport corridor in the north-south direction, known as the Corridor Vc. Favourable geotraffical position and traffic connection constitute factors that have significantly influenced the development of Mostar as municipal, regional and national centre. Figure 1 shows the location of Mostar in relation to the Pan-European transport corridors and transport infrastructure of Bosnia and Herzegovina. The City of Mostar is a traffic intersection of the railroad Sarajevo-Ploče, the main road M17 and future freeway Sarajevo-Ploče. Two development directions are highlighted within the Mostar municipality area: the north-south (the valley of the Neretva River) and the east-west.

Figure 1 Geotraffical position of Mostar

Considering the central functions of the Municipality and region, it is necessary to ensure the development of transport and communications towards all parts of the Municipality and the region, as well as their integration into organized urban network. The Northern part of the urban area of Mostar is a part of the whole which gains ever growing importance and enables taking over of the very important spatial functions. Within the urban zone “Sjeverni logor” it is planned to build a residential and business complex with 3000 residential units [1], Figure 2.

Figure 2 Regulatory plan “Sjeverni logor” Mostar[2]

The “Study of Transport and Communications for the area of the City of Mostar” [3] was drafted in 2010. However, the impact of construction of the residential and business complex within the area of “Sjeverni logor” to the traffic volume was not analysed or taken into account. That was the reason to perform research and analyse the traffic with an increased number of movements within this part of the city.

2 Methodology of research

Since according to the Urban Plan of the City of Mostar it is planned to build a residential and business settlement of 3000 housing units within the area of “Sjeverni logor”, the calculation of production was performed using the following formula [4]:

$$P = 3 \times V - 500 \tag{1}$$

where:

P production;
V number of vehicles.

The starting assumption was that each residential unit will own at least one vehicle. In that case, the number of vehicles would be 3000, and the production would be:

$$P = 3 \times 3000 - 500 = 8500 \text{ (veh/day)} \tag{2}$$

Through the mentioned settlement it is planned to construct a four-lane road which would be connected by a bridge over the Neretva River with west bank, i.e. Kralja Tomislava Street. On the east side, the complex is connected to Maršala Tita Street southern from the intersection "Mostar North". Using the corrected O-D (original – destination) matrix, the traffic on the existing network and the network of planned roads was simulated. German software PTV Visum was used for simulation. The following scenarios were analyzed:

- Scenario "0" – analyses the case of the existing road network of the City of Mostar without construction of the planned residential and business complex "Sjeverni logor", Figure 3.
- Scenario I – analyses the case of the construction of residential and business complex within the area "Sjeverni logor" including planned roads. The complex would be connected by a bridge to Kralja Tomislava Street running through the western part of the city, and on the east side connected to Maršala Tita Street southern of the intersection "Mostar North", Figure 4.
- Scenario II – is the case of construction of additional four-lane road running along the right bank of the Neretva River, connected with an additional bridge to the northern part of the complex, and northern from the intersection "Mostar North" connects with main road M17, Figure 5.
- Scenario III – it is assumed that highway on the Corridor Vc and the South Bypass of the City would be constructed in addition to mentioned roads, Figure 6.

It should be noted that, within the Spatial Plan, it is planned to construct the Northern Bypass of the City of Mostar, which was not considered in this paper. Since mentioned complex is located nearby the intersection at the northern entrance to the city (Mostar North), it is obvious that the construction of the complex will have the greatest impact to this intersection particularly. Traffic volume within the urban zone was observed simultaneously (screen lines G1 and G2). Traffic analysis has been performed for the time sections 2015, 2020, 2025 and 2030 (years). This paper presents the results for the end of the planning period when it is realistic to expect to have all the planned roads constructed. Comparison of the results was presented on the screen lines G1 and G2 within the urban zone, and U3, U4 and U5 at the access roads to the intersection "Mostar North". The results of these simulations are shown in the following figures.

Figure 3 Traffic volume Scenario "0" – without planned residential area

Figure 4 Traffic volume Scenario I – with planned residential area and roads

Figure 5 Traffic volume Scenario II – with new road on the right side of the river

Figure 6 Traffic volume Scenario III – with the Corridor Vc and the South Bypass

3 Results of the research

The results of the research are presented in Table 1. They refer to the last year of the planning period (2030), per previously described scenarios and per examined screen lines on the road network of the City of Mostar.

Table 1 Traffic volume in 2030 on screen lines

	Traffic volume in 2030 [veh/day]				
	City		Intersection "Mostar North"		
Code	G1	G2	U3	U4	U5
Scenario "0"	65.561	30.706	36.442	30.474	11.915
Scenario I	68.385	15.796	40.474	41.075	16.269
Scenario II	62.218	11.318	16.812	21.346	18.614
Scenario III	55.231	9.328	14.640	22.522	15.266

Figure 7 presents a graph with percentage of increase or decrease of the traffic volume on the observed screen lines. On the screen line which goes through the centre of the city G1 it can be seen that in the case of Scenario I the traffic volume increases by 4.3%. In cases of Scenario II and Scenario III the traffic volume is reduced by 15.8% compared to the initial Scenario "0", Figure 7. At the G2 screen line the traffic volume is reduced in all three scenarios and even up to 69.6% in Scenario III. Regarding the screen line U3, the traffic volume in the case of Scenario I increases by 11.1%, while in Scenario II and Scenario III it decreases to 59.8%. The screen line U4, in the case of Scenario I, has increase of the traffic volume by significant 34.8%, while in two other scenarios occurs traffic relieve. The greatest traffic relieve occurs in case of Scenario II (by 30.0% compared to the initial scenario). Finally, on the screen line U5 in all three scenarios the increase of traffic volume occurs and in that case the least favourable is Scenario II. From the results presented in Table 1, it can be concluded that the intersection "Mostar North" will be overloaded following the construction of the planned complex. The reconstruction of this intersection into multilevel intersection would resolve traffic congestion at the intersection itself. However, without construction of the additional roads, the traffic volume on the access roads increases – Scenario I (Figure 7.). In the case of Scenario II, traffic volume relieve occurs on all screen lines observed except on the screen line U3. However, it can be stated that the congestion problem at the intersection of the "Mostar North" only moves to newly projected intersection located on the main road M17.

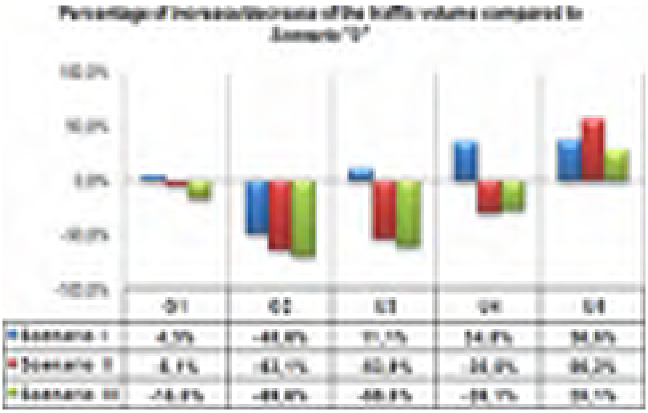

Figure 7 Percentage of increase/decrease of the traffic volume compared to Scenario "0"

Finally, the Scenario III offers traffic volume relieve on the main road M17 and its intersections, because by the construction of the Corridor Vc and the South Bypass the "road ring" around the City of Mostar would be closed while the transit traffic and the part of the original-destination traffic would be moved to these roads.

4 Conclusion

Based on the presented results of the research, it can be concluded that the planned construction of the residential and business complex impacts the traffic volume which should not be ignored. This impact reflects mostly at the intersection "Mostar North". The roads that are planned as part of the residential and business complex affect the redistribution of traffic at the northern entrance to the city. It can also be concluded that traffic congestion can be resolved with:

1 The construction of bridges connecting the planned residential settlement with the right bank of the Neretva River;
2 The construction of the road on the right bank of the Neretva River connecting urban zone with the northern part of the city;
3 The construction of the Corridor Vc and the South Bypass of the city.

References

[1] Službeni glasnik Grada Mostara, Broj 11, pp 1098, 02.11.2012.

[2] Regulacioni plan grada Mostara, http://www.mostargis.ba/presenter/map.jsp, 16.01.2014.

[3] Studija saobraćaja i veza za područje grada Mostara, Građevinski fakulte u Sarajevu, 2010.

[4] Rogers, M.: Highway Engineering, Department of Civil and Structural Engineering Dublin Institute of Technology Ireland, pp. 37, 2003.

[5] User Manual, PTV VISUM 13, Karlsruhe, Germany, 2013. www.ptvgroup.com

[6] PTV VISUM 13, Fundamentals, Karlsruhe, Germany, 2013. www.ptvgroup.com

DETERMINATION OF THE EFFECT OF INTERSECTION CONTROL MODE ON VEHICLE DELAY TIMES

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Abstract

The article deals with the determination of the effect of intersection control modes on vehicle delay times, or with the differences arising from the use of different control modes of traffic lights at low traffic volumes. It describes the methods used in the Czech Republic, and makes a brief comparison with neighbouring countries.In the second part of the article, vehicle delay times on traffic lights are determined using different methodologies and compared with reality. The HCM methodology (method of recording queue lengths in firmly fixed steps) was used and compared with the TP 235 Czech methodology for the calculation of delays and the method of direct measurement of each vehicle delay by a stopwatch. The results are in good agreement; considerable differences occur only in cases where more complex movements are involved (left turn in combination with giving priority to straight direction traffic). Furthermore, based on traffic surveys, vehicle delay times were determined and compared in the mode with traffic lights on (i.e. TL) and after switching over to different TL control modes at low traffic volumes. Different traffic movements at the intersection were monitored at multiple locations for verification and comparison purposes. Based on the results, it is evident that in terms of delay times, any active TL mode used at low traffic volumes is rather counterproductive. Also, environmental impacts (CO_2 emissions, CO, NO_X) tend to grow in the active TL mode. The article also presents the analysis of several model examples using micro analysis software and quantifies the above parameters. The last part brings the initial design of conditions under which TL switching off may be applied. Basic design and traffic engineering characteristics are identified and their effect on the TL control mode is described.

Keywords: traffic lights, HCM, intersection control mode, vehicle delay, environmental impacts

1 Introduction

Delay times of vehicles at intersections represent one of the principal criteria manifesting the traffic quality level in the Czech Republic. Delays at intersections controlled by traffic lights (TL) naturally depend on the control mode (CM) used, and at low traffic volumes, they may cause indignation on the part of drivers who are waiting for the free signal at an “empty” intersection, although they would pass across the intersection without a significant delay in the case of uncontrolled intersection operation. While in peak hours, TL at intersections should be preset to maximize their capacity, at low traffic volumes their setting should significantly differ, so that the cycle is not too long and vehicle delay times are minimized.

1.1 Intersection control modes used in the Czech Republic

There are numerous approaches to traffic control during low demand periods applied both in the Czech Republic and worldwide. They mostly involve uncontrolled traffic or different modes of vehicle actuated signal control, where the traffic signal controller reacts to the arrival of vehicles from different directions assigning accordingly the green light signal. The issue of the operating times of TL has been a controversial topic discussed among traffic experts not only in the Czech Republic, but also abroad, for decades. There are basically two groups of opinions. One group is composed of the proponents of continuous TL operation without exceptions, who prioritize a greater clarity and safety of the TL controlled intersection. Their opponents, on the contrary, prefer differentiated operation of individual TL according to local traffic conditions as a better solution for many reasons.
Different approaches to continuous traffic control using TL in the Czech Republic are evident from the following data coming from four largest cities in the country. (from 3% of continuously controlled intersections in Ostrava to 85% in Prague). Different approaches to the operating times of TL provide numerous opportunities for traffic surveys focusing on the comparison of delay times under different traffic conditions (traffic volumes, traffic control modes).

1.2 Intersection control modes used abroad

The views on traffic control during low demand periods also differ in foreign countries depending on the locality. In Germany, similar diametrical differences among individual cities may be found as in the Czech Republic. Overall, we may summarize them saying that the control mode significantly depends on the age of the used technology. With abilities of modern TL technology the necessity of alternative TL modes tends to lessen.

2 Traffic surveys

2.1 Methods of vehicle delay determination

Three different methods were used to determine the delay times of vehicles at an intersection within the scope of TAČR – TA03030046 project. The HCM method is based on observing the length of vehicle queues at the approach to an intersection in time steps which subsequently serves for deriving the average delay of vehicles. The results are processed using simple relationships that are not the subject of this article and can be found in HCM [2]. For accuraccy improvement 30 minutes interval was used as opposed to the recommendation mentioned in HCM (15 minutes).
The second method is measuring the delay of each vehicle separately with a stopwatch which records the moment when a vehicle stops and the moment when it passes a stop line. This method is considered the most accurate of all.
The third method is the calculation of the delay based on the Czech TP 235 methodology [3]. This methodology was compiled as the average of numerous measurements made at different intersections, mostly in situations approaching the capacity limits of entries to intersections.

2.2 Comparison of methods for identifying vehicle delays

Several intersections situated in different cities (Prague, Brno, Ostrava and Olomouc) were selected for the assessment of the informative value of the results of delay times identified by a traffic survey based on the HCM methodology [2]. The results are compared in Table 1.

Table 1 Delay values comparison

Location			Delay tw [s/veh]		
City	**Intersection**	**Mv.***	**HCM**	**Watch**	**TP235**
Praha	Nárožní – Pod hranicí	^›	7.6	5.4	7.4
Praha	Nárožní – Pod hranicí	‹	27.6	24.9	26.7
Praha	Patočkova – Pod Královkou	‹	25.4	24.3	25.8
Praha	Plzeňská – Jeremiášova	‹	27.0	23.3	9.0
Ostrava	Novinářská – Hornopolní	^›	27.9	29.4	29.0
Ostrava	Českobratrská – Sokolská	‹	47.6	50.9	38.0
Brno	Osová – Jihlavská	‹	37.6	44.5	44.1
Olomouc	Hodolanská – Tovární	‹	46.3	47.9	50.5

* Mv. – Intersection movement

The comparison of eight Czech intersections clearly shows that the HCM-based methodology very well corresponds to the actual delay of vehicles measured with a stopwatch. Comparing the delays calculated on the basis of the TP 235 methodology against the delays measured with a stopwatch, we may also conclude that the methodology is in fairly good correspondence with reality (stopwatch). The exception is measurement No. 4 where the deviation is significant. This is caused by the fact that it is the only approach in the table with the left turn on a separate turn lane being also affected by the opposite direction (oncoming vehicles travel in the same phase). These issues were not elaborated in the Czech TP 235 methodology in an optimum way.

2.3 Comparison of delays in different control modes

Several measurements at intersections controlled by different modes in night hours than during the day was conducted. Due to the extensive amount of data thus obtained, only some selected measurements arre presented in this article.

2.3.1 Fixed control mode vs. flashing yellow control mode

One main and one minor approach were monitored at an intersection in Kladno. Both approaches share a common transfer lane for all directions; the intersection is controlled by a fixed programme with a cycle length of 60 seconds in the daytime, and at 9 p.m. it is switched to the flashing yellow control mode. The delay was measured using the proven HCM-based method from 7:00 p.m. to 9:00 p.m. and with a stopwatch in the flashing yellow mode from 9:00 p.m. to 10:00 p.m. (for the reason of lower traffic volumes). The column Iappr. expresses the vehicle volumes at the monitored approach, while the column Iinters. expresses the total vehicle volumes at all approaches to the intersection.
It is evident from Table 2 that switching this intersection to the flashing yellow control mode was suitably selected. Vehicle delays are smoothly reduced down to minimum values.

Table 2 Survey results – fixed CM vs. flashing yellow CM

City/Inters./Approach	Delay t_w [s/veh]	$I_{appr.}$ [veh/h]	$I_{inters.}$ [veh/h]	Control mode
Kladno Pražská-Unhošťská main approach	9.6	354	1096	fixed
	12.7	268	932	
	9.9	212	686	
	7.5	150	478	
	0.8	116	458	flashing yellow
Kladno Pražská-Unhošťská minor approach	21.9	248	1096	fixed
	21.9	266	932	
	18.6	160	686	
	15.3	118	478	
	13.4	118	458	
	6.3	86	378	flashing yellow

2.3.2 Vehicle actuated control mode vs. flashing yellow control mode

This control procedure is applied at many intersections in the city of Plzeň and at some intersections in Prague. The measurements were always performed 2 hours before switching modes and one hour after switching modes. The delay was measured using the proven HCM method in the actuated mode and with a stopwatch in the flashing yellow mode. It should be noted that the actuated control rates may have varied at the monitored intersections, and the control cycles were also different in length.

It is evident from Table 3 that the vehicle actuated control is not able to provide a substantial reduction in vehicle delay times at low traffic volumes either. After switching to the flashing yellow mode, the delay of vehicles at the monitored approaches sharply decreases at all intersections suggesting that the control had already been inefficient in terms of delays before the mode was switched over.

Table 3 Survey results – actuated CM vs. flashing yellow CM

City/Intersection/Approach	Delay t_w [s/veh]	$I_{appr.}$ [veh/h]	$I_{inters.}$ [veh/h]	Control mode
Plzeň Strnadova-Slovanská minor approach	26.9	108	1394	actuated
	18.6	110	1198	
	23.6	64	1010	
	23.7	62	728	
	9.1	32	646	flashing yellow
	3.8	26	420	
Plzeň Koperníkova-Tylova minor approach	16.1	308	754	actuated
	15.2	210	602	
	19.7	192	612	
	16.2	166	488	
	9.8	108	440	flashing yellow
	8.7	118	358	

2.3.3 Vehicle actuated programme vs. all-red control mode

This control procedure is applied at numerous intersections in Prague and it was also monitored at one intersection in České Budějovice. The measurements were always conducted in the same way as in the previous case. Table 4 clearly shows that if appropriately designed, the all-red control mode may reduce the delay times at an intersection compared to the vehicle actuated control mode. It was, however, identified at the monitored intersections that the majority of vehicles had to stop or at least brake before the intersection as detection devices were not adequately positioned. Although some experts point out the safety factor and the fact that this mode may reduce vehicle speeds in municipalities, it would be interesting to get some feedback from citizens living in the vicinity of such intersections related particularly to noise emisions due to decelerating and accelerating vehicles.

Table 4 Survey results – actuated CM vs. all-red CM

City/Intersection	Delay t_w [s/veh]	$I_{appr.}$ [veh/h]	$I_{inters.}$ [veh/h]	Control mode
Praha Jeremiášova-Radlická	10.7	90	576	actuated
	14.9	70	416	
	10.0	74	350	
	11.5	52	310	
	6.7	62	278	all-red*
	4.1	36	196	
České Budějovice Lidická-Mánesova	34.5	108	542	actuated
	27.0	96	506	
	29.1	84	614	
	26.9	58	294	
	7.0	48	174	all-red*

* all red CM has at all signal groups the red sign in basic state. When a request from one direction comes, it can be changed into the respective phase.

3 Results of traffic simulations

In addition to the impacts on delay times, alternative TL modes at low traffic volumes also affect the majority of relevant traffic engineering characteristics. Environmental impacts of an intersection were compared for the purposes of this article, namely the volumes of vehicle emissions that would differ for different control modes. These emissions were quantified using the Sidra Intersection 6 software. The model configuration used was a symmetrical crossroad with a left turn lane onto the main road and a branch length of 200 m (Fig. 1).

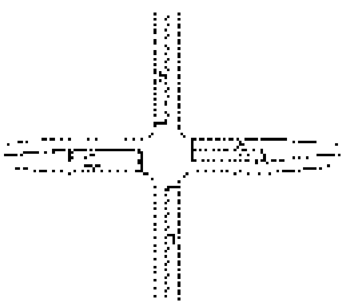

Figure 1 Model design of the evaluated intersection

Three basic control modes were evaluated, a fixed control model with a cycle time t=80 s, an actuated control mode and sign control (E-W major/N-S minor).
To see the effect of approach traffic volumes, two levels of approach traffic volumes were considered, 240 light vehicles+60 heavy vehicles and 600 light vehicles+120 heavy vehicles (for the whole intersection). The distribution into individual directions is uniform, i.e. the traffic loading scheme is also symmetrical for our purposes. The resulting values are presented in Table 5 and Table 6.

Table 5 Model results – 240 light veh/h+60 heavy veh/h

Control mode	Volumes $I_{inters.}$ [veh/h]	Ctrl. delay [s/veh]	Fuel [l/h]	CO_2 [kg/h]	CO [kg/h]	NO_X [kg/h]
TL-fixed	240+60	27.7	10.6	25.4	0.1	0.101
TL-actuated	240+60	13.6	9.7	23.4	0.1	0.096
Stop (2-way)	240+60	10.5	9.1	22.0	0.1	0.091

Table 6 Model results – 600 light veh/h+120 heavy veh/h

Control mode	Volumes $I_{inters.}$ [veh/h]	Ctrl. delay [s/veh]	Fuel [l/h]	CO_2 [kg/h]	CO [kg/h]	NO_X [kg/h]
TL-fixed	600+120	31.1	23.3	56.6	0.2	0.204
TL-actuated	600+120	14.5	21.4	51.3	0.1	0.193
Stop (2-way)	600+120	13.8	20.3	48.7	0.1	0.186

The results indicate the impact of the control mode used on both the expected delay times and the fuel consumption plus related modelled emissions. As it was assumed, this effect is the greatest for CO_2 emissions, which directly rely on the consumption and related vehicle delay times at an intersection. The CO values, however, are practically negligible (in terms of the total volume and its impact), while the drop in NO_x values is reduced by the fact that the volumes of NO_x emissions are only significant at higher speeds, i.e. their volumes in the area of intersections are not so noticeable. In terms of emissions, the difference between the fixed control mode and the stop (2-way) control mode is particularly evident. Here, the drop for CO_2 values is by up to 14%, while for NO_X values the drop is by up to 10% (Table 5, Table 6). Nevertheless, the decrease in delay times in this case is by up to 62% at low traffic volumes. For reference purposes, a simulation for loading with 1200 light vehicles+120 heavy vehicles was also performed which clearly shows that alternative modes of traffic control at intersections lose their effect with increasing approach traffic volumes.

4 Conclusion

As the above results and experience from abroad imply, alternative CMs are only usable under specific conditions which must be unambiguously defined and quantified. These are in particular:
· vehicle volumes and threshold values of switching to alternative CM modes;
· traffic significance of a particular road;
· width layout of individual branches;
· operation of public transport, pedestrian flows and their compositions;
· view parameters (vehicle/vehicle, vehicle/pedestrian);
· coordination of traffic lights;
· accident rates of a respective intersection;
· existing control system (fixed/actuated);
· used TL technology (classic light bulbs vs. LED technology);
· other atypical reasons (traffic calming TL, modifications for the blind, etc.).

The existing situation in the Czech Republic is rather chaotic. Due to the fact that there is no universal regulation specifying control modes, their application and impacts, different approaches are used in different towns. These procedures are frequently in contradiction with the basic traffic engineering knowledge being the cause of negative impacts in terms of both higher delay times and higher noise emissions and environmental burden. Therefore, a clear identification of the basic criteria, different control modes and their impacts on the road network is an indispensable fundamental priority for modern traffic engineering.

References

[1] Webster, N. & Harman, P.: Operation of Traffic Signals during Low Demand periods, Leeds, 2011.

[2] National Research Council (U.S.): Highway Capacity Manual 2010, Washington D.C., Transportation Research Board, 2010.

[3] EDIP s.r.o.: TP 235 – Capacity Assessment of Intersections Controlled by Traffic Lights, Liberec, EDIP s.r.o., 2011 (in Czech).

[4] TSK hl. m. Prahy: Transport Yearbook of Big Cities, Praha, TSK hl. m. Prahy, 2011 (in Czech).

[5] Forschungsgesellschaft für Straßen – und Verkehrswesen e.V. (FGSV), Arbeitsgruppe Verkehrsmanagement: RiLSA – Richtlinien für Lichtsignalanlagen – Lichtzeichenanlagen für den Straßenverkehr, Köln, 2010.

SUSTAINABLE MOBILITY OF SMALL TOURIST PLACES

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Abstract

Article describes the development of a strategic plan of sustainable mobility of the Municipality Lopar on the Rab Island. In small tourist places number of tourists greatly exceeds the number of the local population during the tourist season. Via the integration, participation and evaluation principles existing plans are upgrading. Scenarios of transport system were developed by the analysis of the shortcomings and the introduction of the measures as a whole. Evaluation scenarios were based on ensure the accessibility offered by the transport system to all, improve safety and security, reduce air and nois pollution, greenhouse gas emission and energy consumption, improve the efficiency and cost-effectiveness of the transportation of person and goods, contribute to enhancing the attractiveness and quality of the municipality environment and design. Finally, the proposed measures are necessary, measures to improve, measures of scenarios for each of the reference year to which examines traffic.

Keywords: sustainable mobility, scenario, evaluation, measures

1 Introduction

A new approach of planning mobility in areas, is directed to cleaner and more sustainable means of transport, in particular this applies to pedestrian, cycling and public traffic. This process is based on the existing practice of planning and takes into account the principles of balance, integration, participation and evaluation. In order to effectively apply the new approach and in a large extent, the concepts and tools developed at European level should be adapted to the circumstances of individual member states and then actively promote the national and regional levels [1]. A new approach to urban mobility planning show table 1. Through the planning sustainable mobility, it stimulates the development of urban transport systems to improve the accessibility of urban areas and to ensure the mobility and traffic of high quality to the village, out or inside village [3]. For the attainment of the objective it is necessary planning the urban transport system that tends towards:

- sustainability with consider social, health and environmental component;
- better use of existing urban areas and transport infrastructure;
- improving the urban environment, quality of life and public health;
- improving transport safety;
- reducing air pollution and noise impacts;
- balanced development and integration of different modes of transport.

The planning is encouraged balanced development of all types transport by encouraging towards more sustainable types [3]. The plan features an integrated set of technical, infrastructural and policy measures to achieve these objectives. Integrated planning involves co-operation, coordination and complementarity with local policies, strategies and measures in the field of traffic, spatial planning, land use, health, and social services. The principle of participation implies the involvement of local authorities, citizens, civil organizations and businesses to develop and implement plan to ensure the support and acceptance.

Table 1 Differences between the planning process [1]

Traditional Transport Planning	Sustainable Urban Mobility Planning
Focus on traffic	Focus on people
Primary objectives: Traffic flow capacity and speed	Primary objectives: Accessibility and quality of life, as well as sustainability, economic viability, social equity, health and environmental quality
Modal-focussed	Balanced development of all relevant transport modes and shift towards cleaner and more sustain-able transport modes
Infrastructure focus	Integrated set of actions to achieve cost-effective solutions
Sectorial planning document	Sectorial planning document that is consistent and complementary to related policy areas (such as land use and spatial planning; social services; health; enforcement and policing; etc.)
Short- and medium-term delivery plan	Short- and medium-term delivery plan embedded in a long-term vision and strategy
Related to an administrative area	Related to a functioning area based on travel-to- work patterns
Domain of traffic engineers	Interdisciplinary planning teams
Planning by experts	Planning with the involvement of stakeholders using a transparent and participatory approach
Limited impact assessment	Regular monitoring and evaluation of impacts to inform a structured learning and improvement process

Monitoring and evaluation plan in achieving the defined objectives and the achieved results is implemented through defined indicators.
The basic steps to create a plan sustainable mobility (Figure 1) are presented through the project to encourage sustainable mobility in coastal towns on the example of municipality Lopar. The specificity of coastal cities is seasonal variation in transport demand induced by tourism.

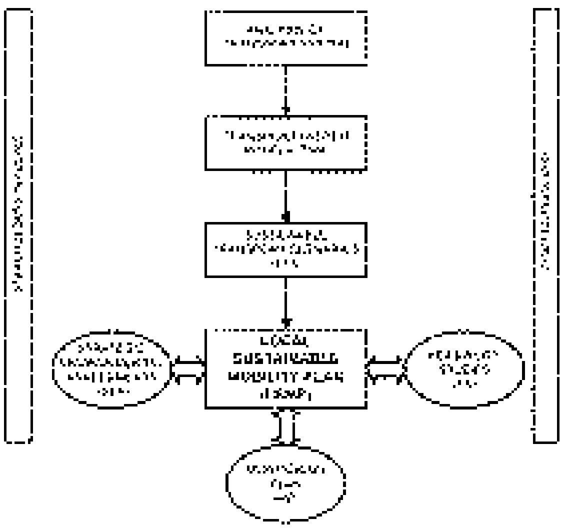


Figure 1 Elements of sustainable mobility planning

Municipalities of Lopar is situated in the Kvarner coast in the northeastern part of the Rab Island, and under the administration of the municipality are the islands of Goli and St. Grgur. Conducted census of people in 2011., in the municipality lives 1,263 inhabitants. In the summer guests visiting or staying in place exceeds 13,000.

2 The components of planning sustainable mobility

2.1 Analysis of the situation

Needs to be analyzed existing transport system and all documentation relate to the transport system and sustainable mobility, [4].
Existing and planned state, county, local and unclassified networks of individual traffic must be detected and displayed in a GIS environment. It is necessary to collect data about transport loads, safety indicators, transport and technical characteristics.
Public bus transport should be described with the stations, routes, timetables, number of transported passengers. Also should analyze the characteristics of ferry and boat lines that connect the place with the mainland. Should be analyzed all ports of county and local significance, offer and functioning of stationary traffic, the number of parking spaces, type and spatial distribution of which is shown in tables and graphs. Need to be determined the length of pedestrian and bicycle paths and their traffic-technical characteristics. In the current planning is implemented:

- analysis of spatial planning documents (spatial plan of the county, spatial plan of the municipality, urban development plan);
- analysis of documentation traffic field (Decision on the regulation of traffic on the territory of municipality Lopar, Decision on conditions and mode of parking, etc.);
- analysis of other documents (Action Plan for development tourism in the municipality Lopar, demographic data, etc.).

Data need be collected and analyzed for all participants in the traffic system. Participants are considered in all institutions, companies, firms and other legal and natural persons who have a particular role or significance in the functioning the transport system, as well as have or may have a role in planning or design transport system.

2.2 Transport system model (TSM)

Transport system model show the current situation and enables the prediction of traffic demand over a period of 15 and 25 years. The model using for detect shortcomings in the transport system, defining and testing scenarios, and as a basis for the development of local mobility plan, feasibility studies and strategic environmental assessment. Steps in the development of transport models:

- collection and analysis the data of the network (table2), demand and calibration;
- development of transport models
  - networks of individual and public transport;
  - zoning system;
  - demand model;
  - joining the traffic on the network.

Table 2 Attributes of network segments

Category	Free flow speed	Permitted direct	Stops
Length	Capacity	Timetable	

The research area is divided into zones. Zones are using for dividing places in less homogeneous surface, connection places with the surrounding area, defining prognostic model and displaying the origin-destination area (Parking locations are defined as separate zones). Table 3 shows frequent zone attribute.

Table 3 Attributes area

Population	The number of parking
Accommodation capacity	Area of attraction

Demand model must be developed on the basis of socio-economic data. Data came from the Central Bureau of Statistics, at Tourist Board the Municipality ofLopar, Spatial Plan and research in the field. After joining the traffic on the network, must be made calibration and validation of the model. The result of a transport model is traffic volume of the overall transport network (Figure 2).

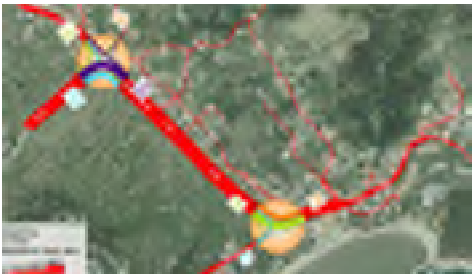


Figure 2 Traffic image of Lopar [4]

2.3 Development of Sustainable transport scenarios (STS)

Problems in traffic and disadvantages in the area should be detected for all transport systems. It should be create scenarios of transport supply as well as the scenarios of traffic demand. Initial baseline scenario serves as a reference level, which is compared with the other scenarios, and allows calculate indicators for evaluating individual scenario.
Selection of the best scenario of the transport system is based on the analysis and evaluation. The proposed scenario should allow for the development of all planned contents and encourage the development of sustainable transport. In the municipality Lopar it is proposed relocation of the access road out of the village and out of the main pedestrian and bicycle corridors. Such measures will improve the attractiveness of places and reduce the impact of noise and exhaust emissions. It is proposed the formation of bicycle and pedestrian network along road corridors but also as independents corridos. Creating listed infrastructure contributes to the increase of number of users and increase safety.

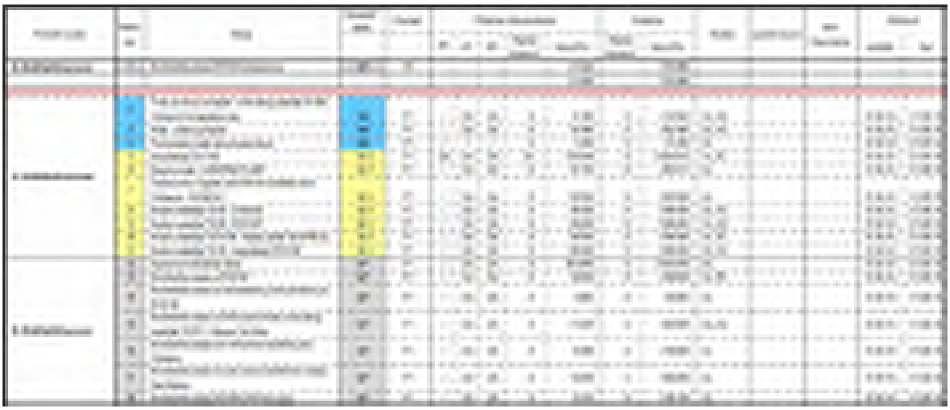
Figure 3 Scenario development of the municipality Lopar [4]

Figure 4 Plan of the cycling network [4]

2.4 Local suistainable mobility plan (LSMP)

On the basis of previous activities and results need be made Local Suistainable Mobility Plan (LSMP). The plan defines strategic objectives, priorities, measures, common and the individual activities. The plan includes traffic problems, environmental problems and the impact of activities on the environment, the ability of actors in the implementation of solutions, the legal framework for the implementation, cost solutions, tools for their implementation and the timeframe of implementation. The municipality of Lopar LSMP contains a total of 60 actions that are associated with the priorities from P1 to P4. Overview LSMP per all mentioned entities and their attribution is given in table 4.

Table 4 LSMP overview [4]



2.5 Feasibility study (FS)

For the chosen scenario of development transport system for which is made LSMP should be made feasibility study. The study includes a cost-benefit analysis for the entire LSMP for a defined period. The study approach to the feasibility from financial, socio-economic and organizational views. At implementation of the proposed measures is mainly case of the indirect effect unless the parking to be charged (direct financial effects). Increased financial effects are expected through:
- increasing tourism capacity;
- increasing parking spaces;
- introduction of tourist bracelets;
- extending the tourist season;
- increasing tourist arrivals because of the attractiveness places.

2.6 Strategic environmental assessments (SEA)

For the chosen transport system should be made strategic environmental assessments. Through strategic environmental assessment, evaluates the likely significant environmental impacts that may arise from the implementation of the plan. The objective of the implementation process is to contribute to sustainable development, i.e. reducing adverse environmental impacts of development activities. The process includes analysis of the likely environmental impact of the development documents, their record in the report and public consultation. At formulating the final plan or program, and the decision on acceptance of the plan, need be participate comments and suggestions on the report.

2.7 Monitoring plan (MP)

For monitoring of implementation of LSMP in the time, organization and implementation of indicators, need to be make the monitoring plan. On the basis of monitoring plan the project manager creates a report, at specific time intervals, about of implementation plan for mobility. The report contains a comparison from planned and realized activities based on defined indicators, deviations in the financial plan, measures to mitigate the difference, suggestions for possible changes to the mobility plan and monitoring plan.
Difficulties in implementing the plan are recognized through monitoring and evaluation what enables timely and effective response. Defining the indicators (Table 5) and target values enables monitoring plan. Through the monitoring plan it is defined:

- connection with the target value indicators;
- data collection strategy;
- monitoring and evaluation plan connected with time during the implementation plan of sustainable mobility;
- responsibility of institutions for monitoring and evaluation;
- minimal involvement of participants for monitoring and evaluation;
- budgetary funds and activities (5% of the budget for the implementation of measures).

Table 5 Indicators

number of passengers on buses	number of tourist arrivals	occupancy of the parking
levels of noise pollution	content of harmful gases	number of accidents (minor, heavy)
length of the bicycle and pedestrian network	amount of traffic on the bicycle and pedestrian networks	number of passengers on a tourist train, tourist boat lines
modal split	number of tourist facilities	tourist bracelets use
yearly average stay of the tourist	rental price	

3 Conclusion

Current practice in planning mobility is not sustainable. A new approach to mobility planning builds on elements which respect sustainable development. Plan for sustainable mobility is based on the principles of balance, integration, participation and evaluation. Through the plan of sustainable mobility it is ensured the need for mobility and development with respect to environmental conditions and future generations. In Croatia, as a young member of the EU, should be encouraged at national, regional and local level, making the sustainable mobility plans based on the framework and tools developed at European level with consideration of local specificity.

References

[1] Guidelines – Developing and Implementing a Sustainable Urban Mobility Plan, European Commission, 2013

[2] Communication from the Commission: “Together towards competitive and resource-efficient urban mobility”, COM(2013) 913 final

[3] Communication from the Commission: “Together towards competitive and resource-efficient urban mobility”, Annex 1, “A CONCEPT FOR SUSTAINABLE URBAN MOBILITY PLANS, COM(2013) 913 final

[4] Traffic study and sustainable mobility plan of the municipality of Lopar, APZ, 2014.

OFFTRACKING CONTROL REQUIREMENTS FOR QUALITY ROUNDABOUT DESIGN

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Abstract

In the last 10 years in Croatia numerous roundabouts have been built in urban and interurban areas. As a rule, these roundabouts replaced the existing three-leg and four-leg intersection in order to increase intersection safety and capacity. Replacement of four and three-leg intersections with roundabouts was often carried out without taking into account the ranking of intersecting roads and spatial requirements. Because of that the capacity and safety levels on these roundabouts were often lower than those on previous four or three-leg intersections. In some cases, poor roundabout design resulted in insufficient lane width on entrance and exit (offtracking control problem). These problems are particularly pronounced when the road axis do not intersect at right angle. Unfortunately, lessons learned from these bad examples weren't adopted, and designers are continuing with poorly designed roundabouts. The main reason for this stems from the fact that there are no official guidelines or regulations for the roundabout design in Croatia. Designers are trying to cope with this situation in different ways; often they are partially studying foreign guidelines and seeking for the solution that fits their problem. In this paper key elements for successful roundabout design will be shown, based on the example of recently constructed roundabout in Croatia, findings from other researchers and international guidelines. Proposed instructions could be used for the development of quality national guidelines for roundabout design.

Keywords: design, roundabout, guidelines, offtracking control, design vehicle

1 Introduction

Professional rules dictate that the designers should take into account certain criteria when designing roundabouts, which include: position in traffic network (urban, rural), spatial limits, traffic flow, intersection capacity and safety. Unfortunately, Croatian designers frequently disregard some of these criteria. It is mainly due to the inexistence of official Croatian guidelines for roundabout design. The other reason is that tenders are frequently won by designers lacking sufficient experience in roundabout design. This paper points to the most frequent mistakes in roundabout design on the example of a constructed intersection.

2 Roundabouts design – Croatian practice

Roundabout design in Croatia is, due to the lack of official guidelines, usually carried out according to the guidelines of the Institute of Transport and Communications [1], then according to German guidelines [2], Austrian guidelines [3] and Swiss norms [4]. When analyzing the mentioned documents it can be concluded that they give similar approach in designing roundabouts which comes down to five steps:

1 designing the intersection elements (islands, lane curbs on entrance and exit, …) and assembling them into an intersection project;
2 offtracking control of the designed intersection;
3 correction of design elements (in case previously designed intersection does not meet offtracking control requirements);
4 sight distance check;
5 fastest path check.

This approach represents the biggest defect of the mentioned guidelines and norms because it is based on a highly iterative procedure which can significantly prolong the designing procedure. What worries is the fact that in the national designing practice intersection design is usually reduced to shaping individual elements. Offtracking control and the correction of design elements is frequently disregarded in spite of many software products on the market which [5, 6] allow very simple offtracking control.

2.1 An example of a poorly designed roundabout

The previous chapter outlined the main reasons for inadequately designed roundabouts in Croatia. The roundabout in the town of Vrbovec constructed in 2013 is an example of this. The construction of this roundabout was an attempt to improve the channelization of traffic flows at the intersection of four streets (Zagrebačka (D41), Bjelovarska (D28), Križevačka (D41), M. Gupca Street) and the entrance to the petrol station (Figure 1). Because of five intersection legs and spatial restrictions the designers had an exceptionally demanding task in front of them. Axes of the existing roads were reconstructed to intersect in the middle of the elevated circular island with the 11.5 m radii. The reconstructed axes intersect at the angles from 37 to 150°. The roundabout has 1.0 m wide truck apron and the 5.5 m wide circular lane. Lane width at intersection legs ranges between 3.1 to 3.9 m, depending on the rank of the road: M. Gupca Street is in the category of street roads, while other streets are categorized as state roads (D28 and D41). Elevated dividing islands are 15.0 to 19.0 m long and were constructed on four legs. In the roundabout zone pedestrian and cyclist traffic is organized with level crossings on all legs.

Figure 1 Plan of the roundabout in Vrbovec

After the roundabout construction and its opening to traffic it was found that, when turning from Zagrebačka and Bjelovarska into Križevačka Street, trailers and semi-trailers of heavy trucks tread with their wheels on the elevated curbs. The basic reasons for that are the following:

- exit lane width was insufficient because offtracking control with design vehicles was not taken into consideration;
- the applied rounding radii of carriageway edges, recommended by design guidelines [1] are applicable at intersections with approach road intersection axes of 90°, but not in the case of the mentioned intersection;
- recommendations of guidelines [8] and norms [3] which refer to the smallest lengths of the arc between the neighbouring legs axes were disregarded (Figure 2).

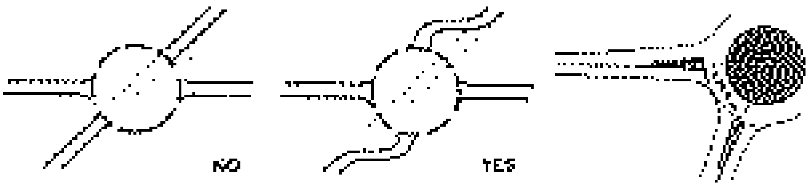

Figure 2 Reconstruction of road axes [8] and the minimum length of the arc between neighbouring leg axes [3]

2.2 Reconstruction solution

After offtracking control, by means of a specialized software [7] for the design vehicle 16.5 m long truck with semitrailer, it was found that the mentioned lane on Križevačka street should be widened by roughly 1.3 m (Figure 3). Although additional offtracking control showed that there is a need for widening the remaining lanes (for the value ranging between 0.1 to 0.5 m) only that critical lane was reconstructed (Figures 3, 4). Additional costs and the waste of time could have been avoided if offtracking control had been conducted already in the design phase.

Figure 3 Offtracking control of the roundabout in Vrbovec

Figure 4 Lane widening on Križevačka Street

3 Proposal of offtracking control at roundabouts

At all intersections including roundabouts safe and unobstructed traffic flow should be secured. That is the reason why in the designing phase it is essential to check the design vehicle's possibility of passing through the roundabout.

3.1 Design vehicle selection

Just as in Croatia there are no official guidelines for roundabout design, there are, equally, no defined design vehicles for offtracking control at such intersections. The valid Croatian norm for the design and construction of intersections at grade (based on standard JUS U.C4.050 from 1990) defines the following design vehicles for offtracking testing conditions: a truck and a truck with trailer [15]. The mentioned norm does not include the rules for designing roundabouts, and the features of design vehicles defined by the norm [15] have not been harmonized with the features of the vehicles defined in the existing legislation of the Republic of Croatia [14]. Namely, the dimensions of vehicles in road traffic in the Republic of Croatia, as well as in other EU member states, have been harmonized with the Council's Directive 2002/7/EC (96/53/EC) [13] through the book of regulations [14] which prescribes the technical categories of vehicles, their dimensions and masses, axial load carrying capacity, devices and equipment that they must possess and the requirements that the devices and equipment of motor vehicles and trailers in road traffic must meet. With regard to the stated facts it can be concluded that in Croatia it is necessary to establish the range of vehicles for the selection of the design vehicle according to the position of the intersection in the road network. The range of vehicles can be established by means of three principles:

1. by statistical data analysis on the presence of a certain group of vehicles (personal vehicles, freight vehicles, truck trailers, buses) and their dimensions on the roads in Croatia, in which the criterion for the design vehicle selection would be the frequency of appearance;
2. by statistical data analysis on the presence of a certain group of vehicles (personal vehicles, freight vehicles, truck trailers, buses) and their dimensions on the roads in Croatia, in which the criterion for the design vehicle selection would be the vehicle dimensions (the selected design vehicle should have bigger dimensions and bigger smallest turning radius than the other vehicles from its group);
3. by selecting a vehicle from the existing range of design vehicles of the discussed guidelines [9, 10, 11, 12] according to the criterion of the biggest width occupied by the vehicles in driving on curves. Here it should be mentioned that German [10] and Serbian [8] guidelines offer the widest range of vehicles and that, unlike other guidelines, take into account garbage trucks and public transport vehicles (articulated bus).

3.2 Offtracking control

Offtracking control should be checked at all intersection legs and for all three directions: straight, right and left (Figure 5).

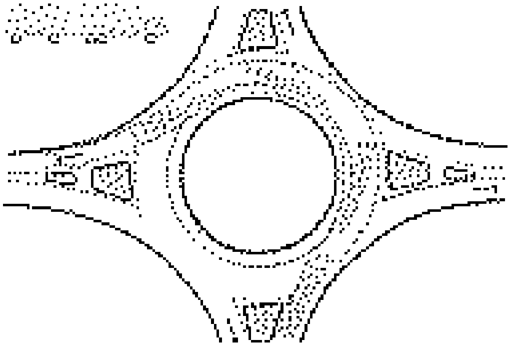

Figure 5 Offtracking control at roundabouts [8]

In order to have a drivable roundabout according to [2, 3, 4, 8] offtracking control should meet the following requirements:
- select a design vehicle which corresponds to the intersection position in the road network;
- ensure the safety lateral width of at least 0.25 m along elevated curbs in the circular lane;
- ensure the safety lateral width of at least 0.25 m along elevated curbs at entrance and exit;
- the design vehicle bus is not allowed to use the paved part of the circular pavement around the central island (truck apron);
- offtracking control should be conducted by design vehicle's templates or verified software.

4 Conclusions

One of the main problems in designing roundabouts in Croatia is the lack of quality design guidelines and frequent dereliction of important design steps such as offtracking control, which cause significant mistakes leading to unnecessary reconstruction expenses as well as the reduced traffic flow safety at intersections. Special attention should be dedicated to offtracking control during the reconstruction of the existing intersections, where larger departures of intersection angles from the right angle lead to additional designing problems. Due to this it is important to clearly establish minimum allowable intersection angles for which the diversion of the approach road axis is not necessary. Since the selection of the design vehicle plays the most important role in appropriate roundabout design, it is indispensable to define the range of the design vehicles and harmonize their characteristics with those of vehicles defined in the existing legislature of the Republic of Croatia.

References

[1] Dadić, I., Tollazzi, T., Legac, I., Čičak, M., Marić, V., Kos, G., Brlek, P.: Smjernice za projektiranje i opremanje raskrižja kružnog oblika – rotora, Institut prometa i veza, Zagreb, 2002.

[2] Merkblatt für die Anlage von Kreisverkehren, K 10000, FGSV (Forschungsgesellschaft für Strassen- und Verkehrswesen), Köln, 2006.

[3] Plangleiche Knoten – Kreisverkehre, Österreichische Forschungsgesellschaft Straße-Schiene-Verkehr (FSV), (RVS 03.05.14), Wien, 2010.

[4] SN 640 263 – Schweizer Norm, Knoten mit Kreisverkehr, VSS, Zürich, 2000.

[5] Vehicle Tracking. Integrated swept path analysis software. http://www.autodesk.com/products/vehicle-tracking/overview

[6] Model, Analyze, and Visualize 3D Vehicle Swept Path Movement. http://www.transoftsolutions.com/autoturnpro

[7] Savoy Computing Services Limited, User Manual for AutoTrack 5, Vehicle Swept Path Analysis, August 2003.

[8] Maletin, M. et al.: Tehnička uputstva za projektovanje površinskih raskrsnica (PGS-PR/07), Građevinski centar, Beograd, 2010.

[9] Plangleiche Knoten – Kreuzungen, T-Kreuzungen, Österreichische Forschungsgesellschaft Straße-Schiene-Verkehr (FSV), (RVS 03.05.12), Wien, 2007.

[10] Bemessungsfahrzeuge und Schleppkurven zur Überprüfung der Befahrbarkeit von Verkehrsflächen, Forschungsgessellschaft für Straßen und Verkehrswesen (FGSV), Köln, 2001.

[11] Maletin, M. et al.: Tehnička uputstva za planiranje i projektovanje gradske putne mreže (PGS-M/06), Građevinski centar, Beograd, 2010.

[12] SN 640 271a – Schweizer Norm, Kontrolle der Befahrbarkeit, VSS, Zürich, 1990.

[13] Directive 2002/7/EC of the European Parliament and of the Council of 18 February 2002 amending Council Directive 96/53/EC laying down for certain road vehicles circulating within the Community the maximum authorised dimensions in national and international traffic and the maximum authorised weights in international traffic, OJ L 67, 9.3.2002.

[14] Pravilnik o tehničkim uvjetima vozila u prometu na cestama, NN 74/09, 51/10, 84/10, 145/11 i 140/13, 2013.

[15] HRN U.C4.050, Projektiranje i građenje cesta, površinski čvorovi, tehnički uvjeti, 1990.

COMPARISON BETWEEN MODELLED AND MEASURED TRAVELLING TIME IN URBAN ROUNDABOUTS

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Abstract

For the analysis of the existing and planned segments of the traffic system, a traffic modelling is used, and the choice of models depends on temporal and spatial limits of a model and the context of application of modelling results. Functioning of a traffic system is under the influence of variable human behaviour. Researches show that the behaviour of a driver is, among other things, territorially and culturally conditioned. Accordingly, there is no universally applicable model, so the adjustment of modelling to local characteristics of traffic system and its users is a necessary prerequisite for the use of any traffic model. The success of modelling of urban traffic networks and its segments is interrelated with successful modelling of critical network segments. According to a number of criteria, the most critical point of every traffic system is the intersection. The simulation models are very useful tool for the analysis of existing critical network segments and prediction of traffic conditions on existing and planned intersections. However, it is questionable whether they can be expected to give realistic modelling results that can be applied in the methodology, analysis and design of intersections in local conditions. Within this paper, a special attention has been paid to the microsimulation modelling of urban roundabouts. Accuracy of modelling results has been established by the comparison of travelling times between measuring points gathered in calibrated and uncalibrated VISSIM microsimulation models and the ones gathered in the field.

Keywords: travel time, microsimulations, VISSIM, roundabouts

1 Introduction

Development of mathematical, mathematical-empirical and simulation models of the traffic system was generated by the need to analyse the impact of the planned facilities and measures, as realistically as possible. Modelling as an analytical tool started to evolve in the forties of the last century. A great acceleration of development of mathematical modelling came about the same time as the rapid computerization and possibilities of solving great systems of equations in real time.

The application of various simulation models needs to be considered within the temporal and spatial scope. Specific simulation models are developed for certain types of traffic analysis and are intended for decision makings which differ in temporal and spatial coordinates. Macroscopic traffic models treat traffic flow as a kind of flow which behaves in accordance with continuum properties. Mesoscopic models incorporate the modelling of individual vehicles movements, where the operating characteristic, such as delays, is modelled in accordance with macroscopic modelling principles through the relationship between speed and density of traffic flow.

A closed driver-vehicle-environment cybernetic system, based on the feed-back relationship in reality, is the most similar with simulation models at the microsimulation level. Today's microsimulation models are able to model the stochastic nature of traffic flow at the multi-modal level: car – truck – bus/tram – cyclist- pedestrian, through a detailed movement modelling of each entity.
Modelling of roundabouts by simulation tool has its own specificities that can lead to significant discrepancies between modelled and measured data [1]. The microsimulation models are unquestionably a very useful tool for the analysis of existing critical network segments and prediction of traffic conditions on existing and planned intersections. However, it is questionable whether they can be expected to give realistic modelling results that can be applied in the methodology, analysis and design of roundabouts in local conditions. The best insight into the reality of the modelling results is provided by a comparison of modelled and measured traffic indicators.

2 VISSIM

VISSIM is a stochastic, discrete, micro-simulated model designed for traffic analyses. It started to develop in Germany at the University of Karlsruhe in the early '70s of the last century. VISSIM traffic model, which enables a detailed analysis and a large number of iterations in real time, is based on testing of various traffic scenarios. It includes empirical and measured data of each examined component of a modelled system and their interactions. Modelling results of each scenario are comparable. They focus on the analysis of alternative solutions, short-term traffic planning or optimization of particular elements of objects and/or evaluation of specific traffic regulation.
The simulation system of the VISSIM model contains two major program components. The first is the model of traffic distribution, and the other is the model of traffic signalization. Every second the main program detects the phase of traffic light signals and based on this data updates a traffic image (second by second, vehicle by vehicle). The stochastic nature of the model lies in the dynamic behaviour of system entities, it is in the function of gathering data from the environment and it is not fully determined by the earlier phase of the simulation.
The difference between VISSIM and other microsimulation models lies in the structure of the network model. Most microsimulation models are based on the node-connector structure, and VISSIM network model is structured on the basis of connectors and links. According to the logic of this kind of a structure, a vehicle that has come to the end of a connector changes the lane (link) and chooses the one that will take it to the desired destination as quickly as possible. The innovative structure allows modelling of complex intersections which reflect a realistic traffic situation. For the longitudinal vehicle movement the model implements the sub-model of psycho-physical modelling of car-following behaviour, and for lateral movement there is the sub-model based on the defined rules of acceptable time gap for changing lanes in vehicle moving [2].
Dynamic elements include the following information: size and structure of traffic flow, location of decision on the route choice, traffic volume distribution, traffic regulation, priority rule, etc. Dynamic elements of the network can be partially adopted from other programs in the form of an OD matrix, data on traffic regulation, main and secondary traffic flows, which shortens the time of framing the network model. Vehicles in the network are defined by the parameters of maximum and desired acceleration and deceleration, speed and desired speed distribution.
The development of computer graphics tools has opened up great opportunities for creating a high-quality three-dimensional presentation of the modelled part of the network or facility. Graphic animation has the ability to change views and perspectives [3]. Graphic presentation and animation are very important tools that help decision makers and wider public visualize and understand specific transport solutions.

3 Measured and modelled travelling time

To check the applicability of the microsimulation tool on roundabouts in local conditions, the two Osijek's single-lane roundabouts were chosen. The first selected roundabout was Vinkovačka–Drinska roundabout, the four-approach intersection of the primary urban network and the second location was a four-approach Kirova-Opatijska roundabout (Figure 1).

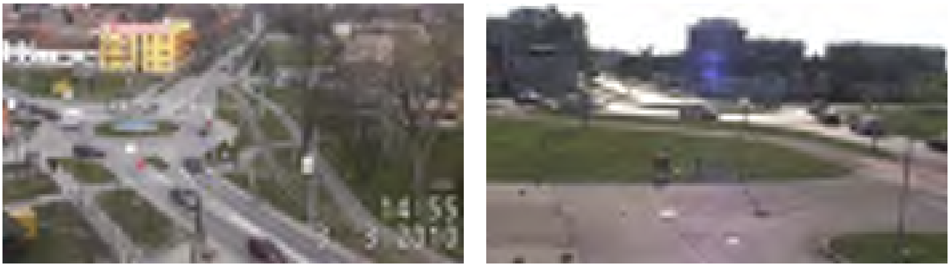

Figure 1 Two single-lane roundabouts in Osijek

VISSIM has developed, and it is continuously developing, a significant number of traffic parameters that can be analysed in traffic modelling, from operational features and economic indicators to environmental impact parameters (noise and air pollution). Measurability in the field was the main criterion for the selection of travelling time as indicator of operational characteristics that will be used as output simulation value of the observed intersection model. Model calibration is the adaptation of a model to local specificities. According to the Highway Capacity Manual, calibration is the process of comparing model parameters with actual data obtained by counting and measuring at a local network [4]. The aim is to reduce the discrepancy between output results of a simulation model and data obtained by measurements and observations in the field. Model validation is evaluation of calibration model efficiency by comparing modelled and measured traffic parameters. VISSIM microsimulation model calibration in local conditions is done by application of neural networks [5]. Identification of influential parameters, the range of their values and optimization of the influencing parameters and their values by some of optimization tools are an integral part of calibration process [6,7,8].
Review of results of counting traffic in the field and the procedure for creating a layout for microsimulation modelling is available in the references [9].
Travelling time for the measured sections for both intersections are measured and compared to the modelled travelling time for the same stream with calibrated and non-calibrated (default) values of model input parameters. The measured values of the observed traffic indicator – mean value of measured travelling time between measurement points are shown in Table 1. Comparison between average value of measured and modelled travelling time is shown in the Table 1.

Table 1 Measured and modelled mean value of travelling time (s)

	Rounadbout 1			**Rounadbout 2**
	1st measurement	**2nd measurement**	**3rd measurement**	**4th measurement**
Measured value	21,8	19,9	18,1	13,3
Default model	20,3	20,3	17,6	13,1
Calibrated model	21,4	19,8	17,6	13,1

For the analysis of travelling time, measuring points were selected in a way that they include one left-turn traffic stream for both observed intersections. VISSIM has the ability to analyse the travelling time of each vehicle in the examined traffic stream between measuring points. At the same time the number of vehicles generated by the model is not necessarily equal to the number of vehicles counted, because the model strictly holds the default traffic distribution and not the traffic load entered into the model. Comparison of each specific measured and modelled travelling time for every vehicle that passed between measuring points for the first measurement (roundabout 1) is shown in Figure 2. In Figure 2 it can be seen that calibrated model provides better matching with measured individual travelling time of observed traffic stream for first intersection.

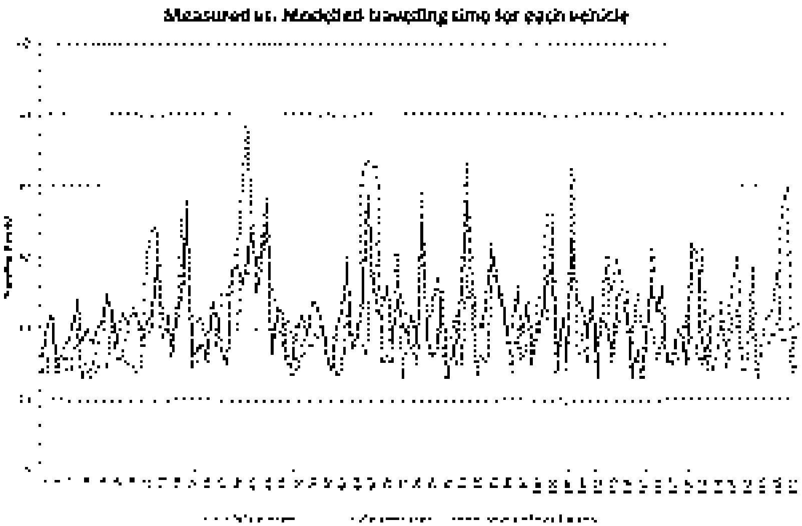

Figure 2 Comparison of modeled and measured traveling time for each vehicle of observed traffic stream (roundabout 1)

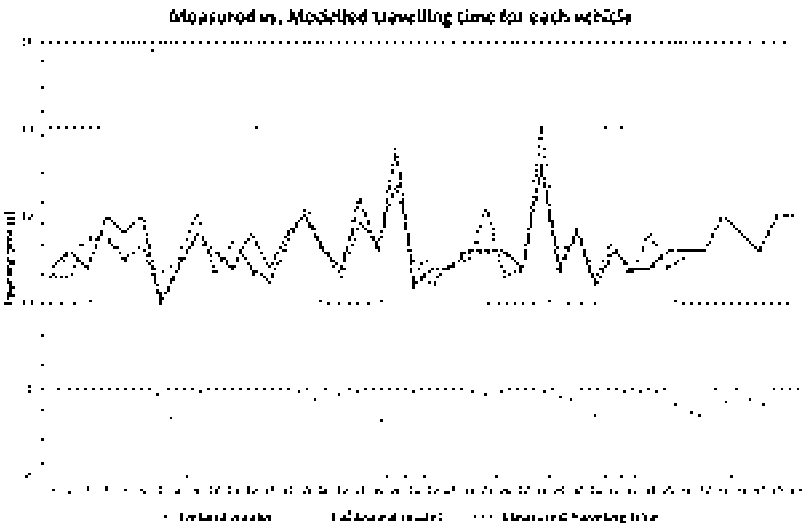

Figure 3 Comparison of modeled and measured traveling time for each vehicle of observed traffic stream (roundabout 2)

In conditions of low traffic load both calibrated and default model provide the same mean travelling time, and even analysis of travelling time of each individual vehicle in traffic stream does not show significant differences, as shown in figure 3.

4 Discussion

A basic insight into the reality of modelling results is obtained by comparison of mean values of traffic indicators acquired from modelling and measuring in the field. For a certain type of traffic analysis, such as selection of type of intersection in early stages of designing, comparison of alternative solutions for an intersection, optimization of intersection shaping elements and analysis of different traffic scenarios, that sort of modelling results is applicable.
Comparison of the mean travelling time obtained from calibrated and uncalibrated models with results obtained from the measuring in the field, clearly shows, as expected, that the calibrated model provides more realistic results, although the differences are not significant. In the first set of measured data, the difference between the travelling time and the modelling results of the calibrated model is 1.8%, and the uncalibrated model has given 6.9% shorter travelling time. In the second set of measured data, the difference between the measured travelling time and the one obtained from the calibrated model is 0.5%, and the uncalibrated model has provided 2% longer travelling time. In case of a low traffic load, both calibrated and uncalibrated model give the same travelling time. In the third set of measured data, the difference between the measured and the modelled travel time is 2.8%, and in the fourth it is 1.5%. In all of the examined cases, calibrated model reaches results which differ from the measured ones by less than 5%, and those are considered to be realistic modelling results [2,6].
Most of the model input parameters, chosen for optimization in the process of calibration [5,8] belong to the set of parameters that reflect behaviour of a driver in local conditions. The fourth set of measured data is made at the other urban roundabout with the idea to check if the calibrated model is applicable only to the roundabout at which the calibration is done or it can be applied to all one lane roundabouts in the examined traffic network. Unfortunately, both models – calibrated and uncalibrated – gave the same results of modelling of travelling time, so the initial hypothesis must be checked by analysing a new set of measured data and additional traffic indicators that are measurable in real traffic conditions, as well as the car-following parameters (mean and maximum length at the entrance to the intersection and the number of vehicles stopping at the examined entrance), delays, etc.
For more detailed and more sensitive analysis, such as traffic safety modelling on the micro-simulation level, identification of potential critical points according to traffic safety criteria, modelling of noise and air pollution, fuel consumption, etc. the mean values of traffic indicators are too rough and are usually not applicable. The advantage of microsimulation models is the fact that they model the movement of each entity separately and that they model the interactions of those entities through defined rules of priority and modelled response time which is variable. For such analysis, not only spatial, but also temporal distribution of traffic is important, so the additional calibration should be done by changing the value of a random number generator (input model parameter – random seed). Thus, the modelled temporal traffic distribution is approaching the real temporal traffic distribution, which has an additional impact on the reality of modelling results.

5 Conclusion

To make a certain traffic model applicable in analysis of a particular traffic network and its users, it needs to undergo calibration and verification processes so that it can model the actual traffic system with sufficient reliability. After the required reality of simulation models of the actual traffic system is achieved, they can be used for simulation of expected traffic growth, economic analysis and some other modelling parameters of the future traffic system and infrastructure.
Accuracy of modelling results is established by the comparison of values of traffic indicators gathered from modelling and the ones gathered in the field. Comparison of travelling times between measuring points in selected urban roundabouts shows that the calibrated model gives results that differ from the measured values of the travelling time by less than 5%, and such modelling results are considered to be realistic.
In order to check the applicability of the calibrated VISSIM microsimulation model to some other one lane roundabout in the examined urban traffic network, it is necessary to introduce additional parameters in the traffic analysis, such as the parameters of car-following, delays, etc. At the other examined roundabout, both models (calibrated and uncalibrated) gave the same modelled travel time between measuring points, which differs from the measured one for only 1.5%.

References

[1] Trueblood, M. & Dale, J.: Simulating Roundabouts with VISSIM, 2nd Urban Street Symposium, Anaheim, 2003.

[2] Kim, S.J.: Simultaneous calibration of a microscopic traffic simulation model and od matrix, Ph dissertation, Texas A&M University, 2006.

[3] Tonndorf, J. M. & Vorotovic, V.: Visualisation of Traffic Behaviour Using Computer Simulation Models, Journal of Maps, Vol. 2007, pp.135-148, 2007.

[4] Highway Capacity Manual 2000, Transportation Research Board, National Research Council, Washington D.C.

[5] Ištoka Otković, I., Tollazzi, T. & Šraml, M.: Calibration of microsimulation traffic model using neural network approach, Expert systems with applications, 40, pp. 5965-5974, 2013.

[6] Park, B., Won, J. & Yun, I.: Application of microscopic simulation model calibration and validation procedure: a case study of coordinated actuated signal system, Journal of Transportation Research Record, TRB, National Research Council, Washington D.C., pp. 113-122, 2006.

[7] Park, B. & Qi, H.: Development and Evaluation of Simulation Model Calibration Procedure, 84th Annual Meeting, Transportation Research Board, Washington D.C., 2005.

[8] Ištoka Otković, I., Tollazzi, T. & Šraml, M.: Analysis of the influence of car-following input parameters on the modelled travelling time, Tehnički vjesnik, 5 ; pp. 919-925, 2013.

[9] Ištoka Otković, I.: Using neural networks in the process of calibrating the microsimulation models in the analysis and design of roundabouts in urban areas,Ph dissertation, University of Maribor, 2011.

IDENTIFICATION OF AT-GRADE INTERSECTIONS CHARACTERISTICS FOR DEFINING BASIC INPUTS INTO MCA METHODOLOGY

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Abstract

The article deals with the determination of basic traffic engineering and measurable urban planning criteria needed for a complex assessment of intersections. The current assessment method of intersections in accordance with applicable regulations is virtually bound on suitable capacities but each intersection may also be described by means of a set of clearly expressed values; based on these values, its suitability for given conditions and thus also the utility value of a proposed design solution may be determined by means of the Multi-Criteria Assessment (MCA) application. The basic criteria defining each at-grade intersection were the mean delay time, the safety index, environmental impacts (particularly CO_2, NO_X and PM), noise impacts on the surroundings, construction and operating costs. In this context, it is obvious that it is also necessary to define the input parameters of a respective intersection on the basis of which the above values of the criteria will be defined. The article primarily describes the dependences between the above criteria and input traffic volumes for basic shapes of intersections. In the first phase, basic layout types of intersections were modelled in the form of a set of basic shapes of intersections – in the future, we plan to extend this set by multi-lane intersections and intersections with separate turn lanes. The distribution of traffic volumes into individual branches was carried out for several basic combinations. The dependences are elaborated in a clear graphical and tabular form allowing the designer or investor to make a preliminary assessment of different design options of an intersection, particularly in the initial phases of design. It is also possible to use the dependences as an alternative to computerised assessment in accordance with applicable regulations.

Keywords: multi-criteria assessment, intersection, intersection capacity, environmental impacts, noise impacts

1 Introduction

The performance and quality of the road network is primarily affected by its design elements. In urban areas the apparently critical limiting element are intersections. In this context, an adequate design of intersections determines all traffic engineering characteristics of the network. The design of intersections is obviously regulated by a set of technical criteria that must be observed in the design. At the same time, virtually each developed country has its own applicable set of technical and legislative regulations that describe and, to some extent, limit the design process. The article describes the method of obtaining rough input data for all relevant characteristics affecting the design of an intersection. The final values may subsequently serve as the basic input for the multi-criteria analysis of the respective design solution or for the recalculation and subsequent operational and economic assessment according to a relevant methodology in force (HDM-4).

2 Methodology of defining inputs for MCA

2.1 Existing assessment procedures of intersections

Comparing the design and assessment methods used e.g. in Europe, it comes out at a closer look that while the technical and design parameters of projects differ only marginally, the assessment of traffic engineering data is performed using a relatively large scale of methods that are not always ideal. The majority of such assessments do not take into account running costs, or potential environmental and noise emissions which, however, have a relatively large impact on the surroundings, especially in high-density developments. The current design practice is such that an intersection is evaluated in traffic engineering terms by its capacity (LOS), while the priority in terms of investment are construction costs.

The assessment in terms of capacity based on applicable regulations is relatively simple. A major problem, however, is that according to conducted surveys the results reached during this assessment quite often differ from reality. Moreover, different methodologies which do not provide comparable results when compared against each other frequently apply in different European countries. Therefore, due to very similar drivers' behaviour across Europe, it is clear that the applied methodologies often work under ideal conditions that, however, rarely occur in practice.

Safety of intersections is also practically not assessed numerically or analytically based on predictive models. The absolute majority of safety audits and assessments are related to the existing situation, i.e. the moment when the respective intersection has already been assessed. In conjunction with the fact that for a valid safety assessment, the respective intersection must be in operation for min. 10 years (some sources even say 20-25 years), it is clear that the current situation is not ideal.

Construction costs are of interest particularly to the investor. In terms of the operational and economic assessment, however, it is evident that construction costs account for only a relatively small proportion of the total funds spent on the intersection. Thus, neglecting running costs of a designed solution is a fatal mistake in the long-term perspective.

2.2 Traffic engineering and economic criteria

In the context of traffic engineering criteria, it must be emphasised that these criteria lend themselves to unambiguous quantification and comparison. Also, there are numerous other factors playing some role in the design that may affect the result, such as the existing configuration, coordination with other intersections, etc. The following basic criteria were identified:

- intersection capacity (mean delay time);
- intersection safety (safety index);
- vehicle emissions (emissions of CO_2, NO_x, PM);
- noise emissions (average equivalent noise level L_{dvn});
- construction costs (€);
- running costs (€/hour).

The Aimsun micro simulation software was used to identify the traffic engineering characteristics and emissions for individual types of intersections. An integral part of the project was also the software calibration to the conditions valid in the Czech Republic and the subsequent verification of the calibration. The safety of an intersection was identified on the basis of exact research conducted in the conditions of the Czech Republic, while construction and running costs were identified using applicable methodologies in force in the Czech Republic (URS data base, HDM-4).

2.3 Assessed shapes of intersections and distribution of traffic volumes

The basic types of intersections were identified in accordance with the Czech norms, which define the following shapes to be potentially used for newly built and reconstructed intersections (Fig. 1):
· crossroad (4-leg intersection) with/without TL;
· T – intersection (3-leg intersection) with/without TL;
· single – lane roundabout;
· multi – lane roundabout (different types of turbo – roundabout).

Only basic types of intersections were modelled in the first step. The set of intersections will be further extended and the most frequent configurations situated in the road network will be modelled. The next step was the determination of approach traffic volumes and their distribution. It is, however, impossible to consider any arbitrary approach traffic volumes in individual branches as the number of permutations would be too high. For this reason, basic configurations of the traffic volume distribution into intersection branches were selected (Fig. 2) plus two basic compositions of traffic volumes according to the numbers of heavy vehicles in the main-minor road distribution (4%-4% / 15%-8%). The basic types of distribution are:
· 1:1:1:1 (intersection movements in the ratio 1:2:1);
· 1:2:1:2 (intersection movements in the ratio 1:2:1);
· 1:3:1:3 (intersection movements in the ratio 1:2:1);
· 2:3:1:4 (intersection movements in the ratio 2:3:1);
· 1:2:2:1 (intersection movements see Fig. 2).

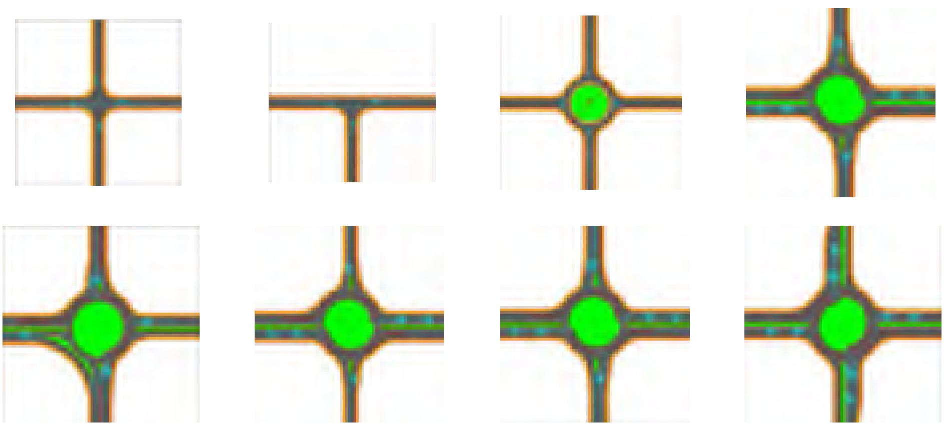

Figure 1 Basic set of intersection types

Traffic volumes will be defined by both the total traffic volume that an intersection should transfer in peak hour and by the traffic volume distribution. The range of traffic volumes selected was 1000 – 3200 veh/h in steps of 200 veh/h.

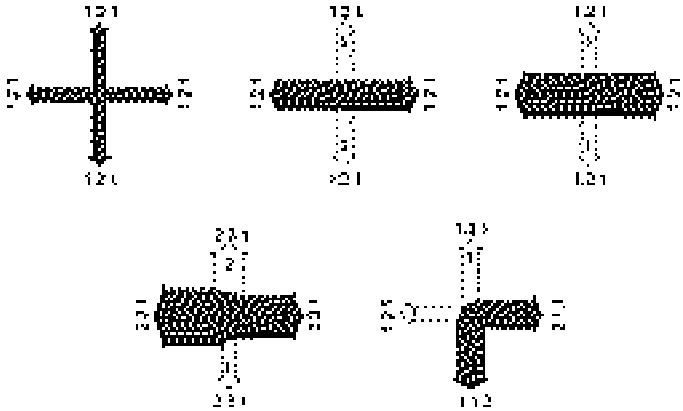

Figure 2 Traffic volume distribution

3 Identification of input data for MCA in practice

3.1 Identification of traffic engineering data by means of micro simulations

All traffic engineering values for the above shapes of intersections and expected distributions of traffic volumes were identified by means of micro simulations. For the purposes of this article, a model output for a single-lane roundabout with a diameter of 40 meters related to total traffic volumes will be presented. The estimated configuration of the traffic volume distribution into intersection branches is in the ratio 1:3:1:3 (intersection movements in the ratio 1:2:1) with 15%/8% of heavy vehicles. The values obtained from the micro simulation model are displayed in Table 1.

Table 1 Micro simulation results for a single-lane roundabout

Volumes [veh/h]	Avg. delay [sec]	CO_2 [kg]	NO_X [kg]	PM_{10} [kg]	$PM_{2,5}$ [kg]	L_{dvn} [dB]
1000	16.4	199.9	0.76	0.049	0.030	61.39
1200	19.1	239.0	0.90	0.059	0.035	62.06
1400	22.2	277.8	1.04	0.068	0.041	62.69
1600	26.7	318.5	1.20	0.078	0.047	63.28
1800	35.0	359.1	1.34	0.088	0.053	63.79
2000	45.9	399.7	1.50	0.098	0.059	64.33
2200	69.5	441.9	1.65	0.108	0.065	64.86
2400	447.0	508.5	1.92	0.123	0.074	65.65

It is evident from the results that the relevant interval of the solution is in the range of 1000 – 2200 veh/h. Values below 1000 veh/h need not be addressed in practice as the growth in delay times at uncontrolled intersections at such low traffic volumes is linear, while at higher traffic volumes (2400 veh/h and above) the busiest approaches already get congested, which leads to an unacceptable rise in average delay times. For better clarity, the growth in the given parameters may also be displayed visually by graphs in Fig. 3 and Fig. 4.

The above table and enclosed graphs clearly show the growth pattern of individual criteria with respect to input traffic volumes. In the case of average delay, the growth in the delay time for traffic volumes between 200-1600 veh/h is practically linear, while from ca 1600 veh/h the growth in average delay starts to be more rapid.

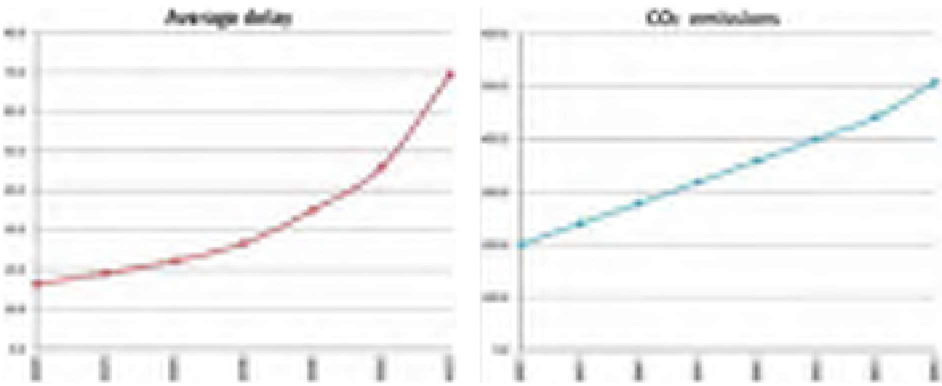

Figure 3 Graphs of average delay (and CO_2 emissions) related to volumes

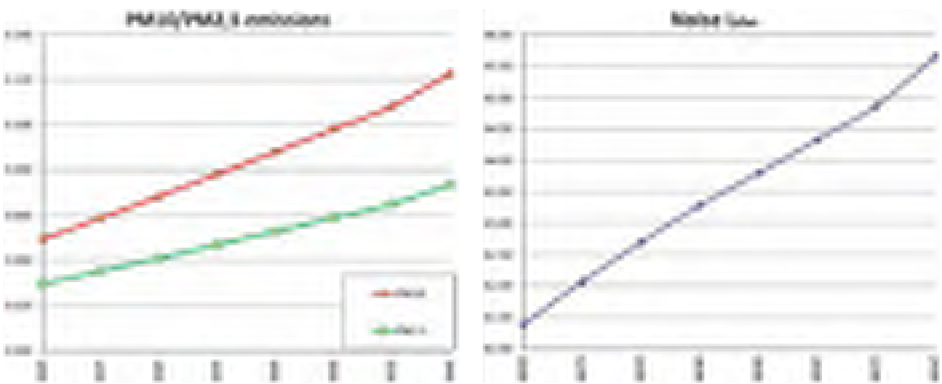

Figure 4 Graphs of PM (and noise emissions) related to volumes

The growth of emissions of individual exhaust gases and noise emissions is linear, as expected, but once the limit capacity of an intersection is exceeded, there is also a steeper rise of individual criteria. In fact, however, this is a limit case as the intersection should be designed so that a similar situation does not occur there.

3.2 Safety index identification

The safety of individual design solutions cannot be practically identified using micro simulation models. Although there are procedures based on traffic models and taking over data from them (e.g. SSAM), these are predictive models that have not been realistically verified in practice and their results are, therefore, not applicable in European conditions. Safety was identified on the basis of exact research conducted in the Czech Republic defining the safety of an intersection by means of a so-called safety index (I_S). The safety index is based on two principal parameters that must be expressed mathematically:

- accident index (I_A) – based on accident statistics from individual already constructed intersections;
- design safety index (I_C) – this index is based on basic design and traffic engineering principles. Each intersection has a certain type of collision points and collision flows where collision movements occur – if the risk rates of such movements may be identified and the relationship between the number of collision points and actual accidents derived, the respective safety index may subsequently be determined.

The considered ratio of both partial indices is 65:35 (1).

$$I_S = 0.65 * I_A + 0.35 * I_C \quad (1)$$

The results for individual types of intersections are displayed in Table 2. We must, however, emphasise that these are initially determined indices, and both the range of their values and the values themselves will be further modified as research continues.

Table 2 Safety index based on at-grade intersection type

Intersection type	Statistics [RN]	I^A [pts]	Conflict points	I_C [pts]	I_S [pts]
T-intersection	0.8	7.5	8	8.0	7.7
Crossroad	1.2	1.0	24	1.0	1.0
T-intersection+TL	0.7	10.0	3	10.0	10.0
Crossroad+TL	1.2	8.0	8	3.4	6.4
RA – 1/1	0.7	10.0	4	9.8	9.9
RA – 2/2	1.2	3.0	16	2.0	2.7
TRA – turbo/egg	1.0	5.0	12	5.4	5.1

3.3 Construction and running costs

Construction costs were determined based on the applicable budget methodology using the URS price system. The final price for the intersection without adjacent sidewalks, considered as newly built, is € 207,895. The price includes only construction works, excluding the costs of traffic engineering measures, relocations of existing networks, etc. Running costs were determined following practical instructions for the assessment of the cost-effectiveness of projects (Czech Road and Motorway Directorate). Running costs and their growth are evident from Fig. 5. Once the intersection capacity is exceeded, there is a noticeable increase in delay times and thus a steep rise in running costs.

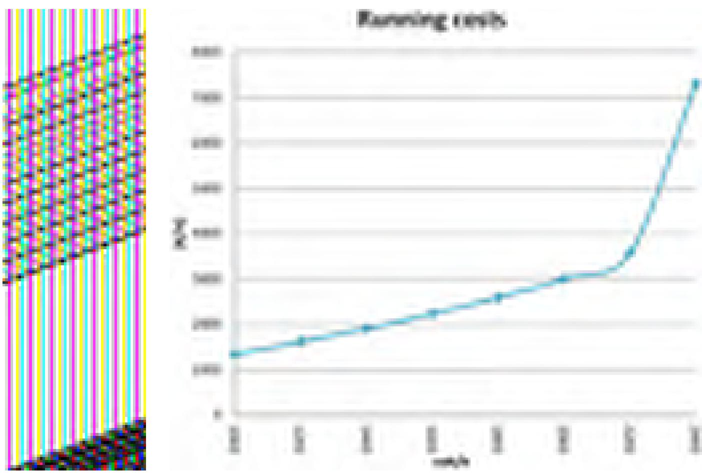

Figure 5 Running costs and their growth

4 Future applications of results

All the above outputs offer relatively interesting application possibilities. Intersections are described using a complex set of data taking into account and quantifying the properties of an intersection in order to assess the designed or existing solution. The basic application domain, therefore, is the application of the results in the subsequent MCA-based assessment of intersections with clearly declared weights of individual criteria. Another option is a purely economic comparison – practically all the above criteria may be expressed in financial terms. Last but not least, the results may be used in design and traffic engineering practice for the purpose of rough estimates of the characteristics and impacts of a proposed solution. Thus, the data offer a relatively wide scope of applications, although with respect to input data, their indicative nature must be taken into account. The identified values will never replace a complex micro simulation model, which, however, is only used in the subsequent phases of project documentation.

References

[1] ČNI, ČSN 73 6102 – Design of road intersections, ČNI, Praha, ČR, 2001.

[2] Slabý P.: Issues of objective evaluation of road traffic safety, project GA ČR 103/02/0065, FSv, ČVUT v Praze, 2005.

[3] Uhlík, M., Hradil, J.: Design and assessment of alternative solutions of multi-lane roundabouts, Silniční obzor. 2011, Vol. 72, No. 7/8, pp. 210-213. ISSN 0322-7154, 2011.

[4] Department for Communities and Local Government, Multi-criteria analysis: a manual, Communities and Local Government Publications, ISBN 978-1-4098-1023-0, 2009.

[5] Federal Highway Administration, Surrogate Safety Assessment Model and Validation : Final Report, 2008.

4 ROAD PAVEMENTS

PAVEMENT MAINTENANCE PROGRAMMING CONSIDERING THREE OBJECTIVES: MAINTENANCE AND REHABILITATION COSTS, USER COSTS, AND THE RESIDUAL VALUE OF PAVEMENTS

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Abstract

This paper presents the development and implementation of a Multi-Objective Decision-Aid Tool (MODAT) tested with data from the Estradas de Portugal's Pavement Management System. The MODAT uses a multi-objective deterministic section-linked optimisation model with three different possible objectives: minimisation of agency costs; minimisation of user costs; and maximisation of the residual value of pavements. The MODAT also uses the deterministic pavement performance model used in the AASHTO flexible pavement design method that allows the gap between project and network management to be closed. The application of the new Decision-Aid Tool is illustrated with a case study involving part of the main road network of Portugal. The "Knee point", which represents the most interesting solution of the Pareto frontier, corresponds to an agency costs weight value of 4%, a user costs weight value of 95% and a weight value of 1% for the residual value of pavements, demonstrating that user costs, which are generally much greater than agency costs and the residual value of pavements, dominate the decision-making process.

Keywords: multiple objective analysis, optimisation models, decision support systems, highway maintenance, pavement management

1 Introduction

In the literature related to pavement maintenance management, only few applications have made use of multi-objective optimisation techniques [1-3]. None of these multi-objective optimisation models considers the minimisation of user costs and a pavement performance model also used for pavement design which allows closing the gap between project and network management. In addition, none of these multi-objective optimisation models considers the maximisation of the residual value of pavements at the end of the planning period which is very important for highway agencies. Greater residual value of pavements is directly related to a greater residual life of pavements which means lower maintenance and rehabilitations costs in the next planning period. This paper presents the development and implementation of a Multi-objective Decision-Aid Tool (MODAT) which considers three different objectives, the minimisation of maintenance and rehabilitation costs, the minimisation of user costs, and the maximisation of the residual value of pavements at the end of the planning period. The MODAT is tested with data from the PMS used by the main Portuguese concessionaire (Estradas de Portugal, S.A.), the institution that acted until 2007 as the Portuguese Road Administration [4, 5].

2 Multi-Objective Decision-Aid Tool

The Multi-Objective Decision-Aid Tool (MODAT) consists of the components shown in Figure 1: the objectives of the analysis; the data and the models about the road pavements; the constraints that the system must guarantee; and the results.

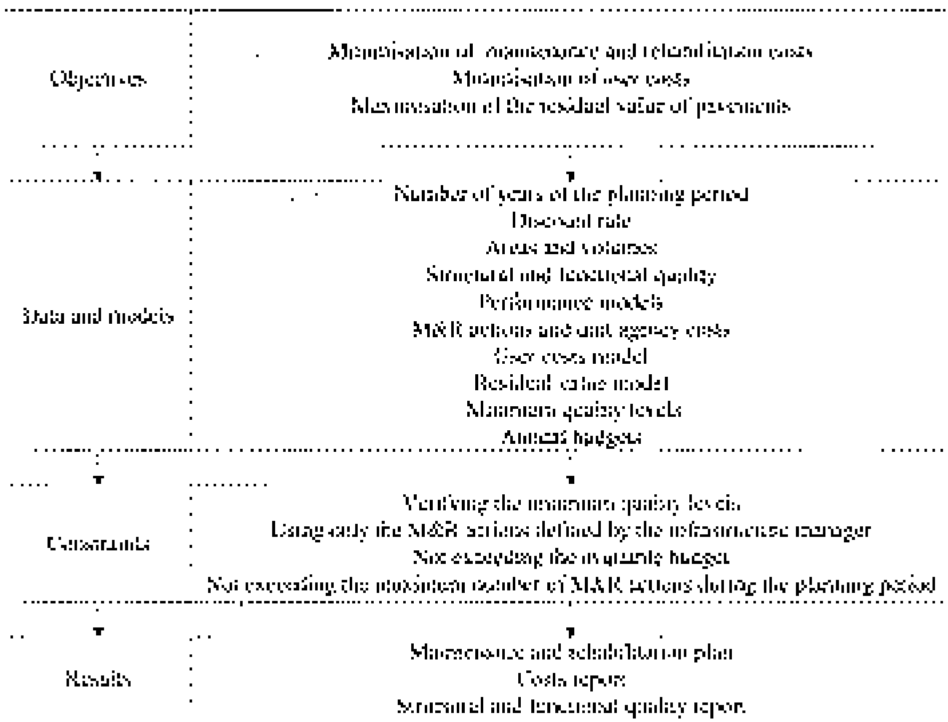

Figure 1 MODAT components.

Several objectives can be considered in the analysis, including the minimisation of maintenance and rehabilitation costs, the minimisation of user costs, the maximisation of the residual value of pavements at the end of the planning time-span, etc. The results of the application of the MODAT to a road network consist of the M&R plan, the costs report, and the structural and functional quality report. Details about the data, the models, and the constraints that the system must guarantee are described in recent published papers in international journals [6, 7].

3 Case study

The MODAT was tested with data from the Estradas de Portugal's Pavement Management System to plan the maintenance and rehabilitation of the road network considering three objectives. The MODAT was applied to the road network of the district of Castelo Branco, one of the 18 districts of Portugal. This road network has a total length of 589.9 km and the corresponding network model has 32 road sections. The discount rate considered in this study was 2.5%. The solutions of the optimisation problem were shown in a 3D representation using MATLAB. Figure 2 presents the three-dimensional (3D) Pareto optimal set of normalised solutions in the objective space by varying the weight values. The "Knee point" was obtained considering the following weight values: $(w_{AC}, w_{UC}, w_{RV}) = (0.04, 0.95, 0.01)$; and it corresponds to the following objective values (AC, UC, RV) = (€69228291.7, €1497083878.6, €37118050.1). The range of values for the three objective functions is (AC_{min}, AC_{max}) = (€44.2x10^6, €206.0x10^6), (UC_{min}, UC_{max}) = (€1424.2x10^6, €2529.3x10^6) and (RV_{min}, RV_{max}) = (€10.9x10^6, €39.2x10^6). Here, w_{AC}, w_{UC}, and w_{RV} are the weight values for each objective function; AC, UC, and RV are the individual objective function values that depend on the decision variables values; AC_{min}, UC_{min}, and RV_{min} are the minimum values obtained for each objective; AC_{max}, UC_{max}, and RV_{max} are the maximum values obtained for each objective.

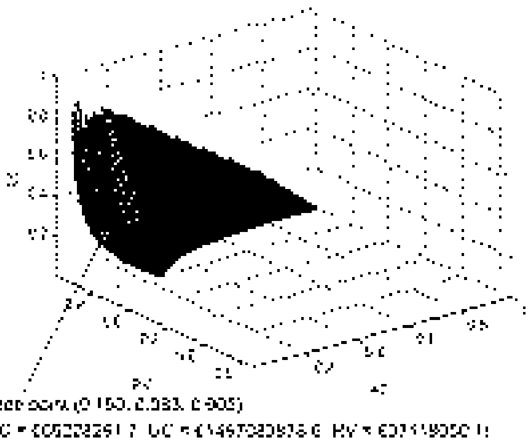

Figure 2 3D Pareto optimal set of normalised solutions.

The final best-compromise solution from the Pareto optimal set of solutions in multi-objective problems is always up to the decision maker. For that purpose, five different M&R solutions of the Pareto frontier were considered for comparison:

a Solution I: Multi-objective optimisation approach (corrective-preventive) considering the “Knee point” (w_{AC} = 0.04, w_{UC} = 0.95, w_{RV} = 0.01);
b Solution II: Multi-objective optimisation approach (corrective-preventive) considering the following weights (w_{AC} = 1.00, w_{UC} = 0.00, w_{RV} = 0.00);
c Solution III: Multi-objective optimisation approach (corrective-preventive) considering the following weights (w_{AC} = 0.00, w_{UC} = 1.00, w_{RV} = 0.00);
d Solution IV: Multi-objective optimisation approach (corrective-preventive) considering the following weights (w_{AC} = 0.00, w_{UC} = 0.00, w_{RV} = 1.00);
e Solution V: Multi-objective optimisation approach (corrective-preventive) considering the following weights (w_{AC} = 1/3, w_{UC} = 1/3, w_{RV} = 1/3).

The costs and normalised costs during the entire planning time-span for these five Pareto optimal solutions are summarised in Figures 3 and 4, respectively. Figure 3 shows that, as expected, solution I (“Knee point”) is the Pareto optimal solution with the lowest total costs (M&R costs, plus user costs, minus residual value of pavements), which was the objective considered in the multi-objective optimisation model. Solution III, considering the weights (w_{AC} = 0.00, w_{UC} = 1.00, w_{RV} = 0.00), is the second best solution, which corresponds to the minimisation of user costs. It is interesting that solution II, which corresponds to the minimisation of agency costs, is the worst solution in terms of total costs. Solution V, considering equal weights for the three objectives, is an interesting solution for the road administration because it has the lowest value of M&R costs minus residual value of pavements.

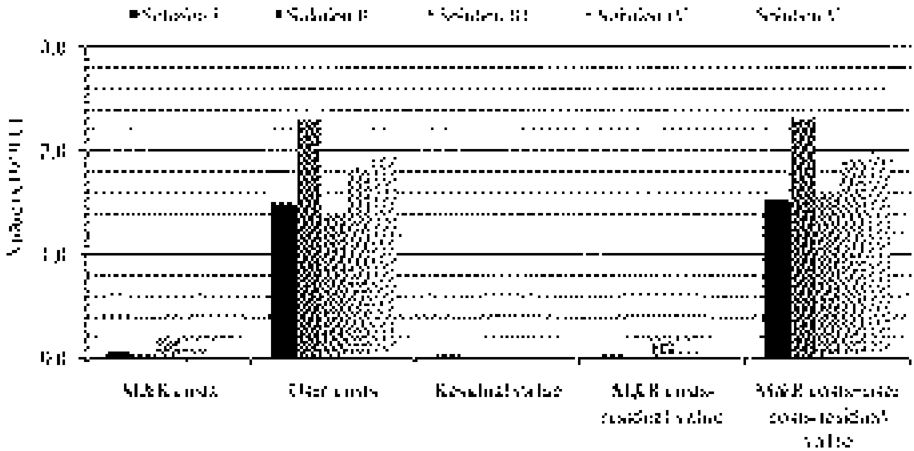

Figure 3 Costs throughout the planning time-span of 20 years.

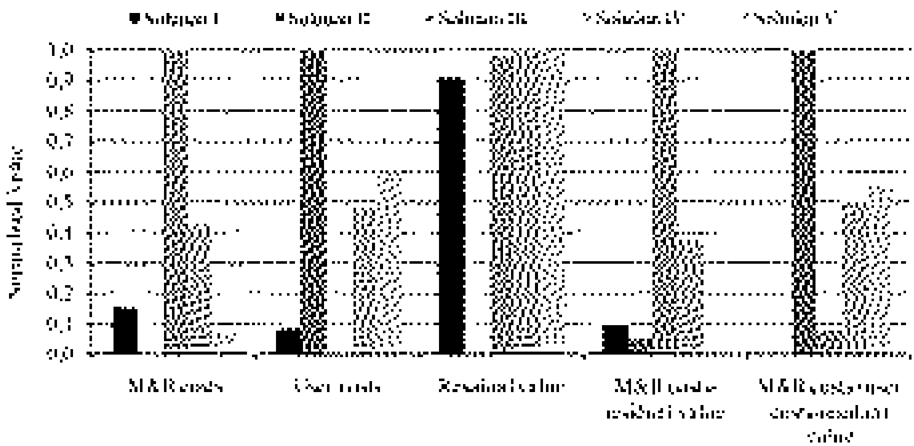

Figure 4 Normalised costs throughout the planning time-span of 20 years.

Figure 5 presents the predicted PSI average value over the years of the planning time-span for all the road network pavements and for each solution. One can conclude that solution III, i.e. the solution of the multi-objective optimisation approach considering the weights ($w_{AC} = 0.00$, $w_{UC} = 1.00$, $w_{RV} = 0.00$), corresponds to the highest average PSI values, as expected, because this solution corresponds to the minimisation of the user costs. Solution I ("Knee point") is the second best solution in terms of average PSI values, also as expected, because this solution corresponds to a high weight value for user costs and small weight values for the other two objectives ($w_{AC} = 0.04$, $w_{UC} = 0.95$, $w_{RV} = 0.01$). As expected, solution II, which corresponds to the minimisation of agency costs, is the worst solution in terms of average PSI values.

The results presented above were defined at network-level. At project-level, the MODAT provides extensive information about the M&R strategy to be implemented for each road section. To analyse these road section-linked results, four road sections were chosen with different attributes in the present year. Table 1 shows the attributes of these four road sections including their present PSI value. Table 2 presents the M&R operations to be applied in the four road sections, considering the five M&R solutions of the Pareto frontier.

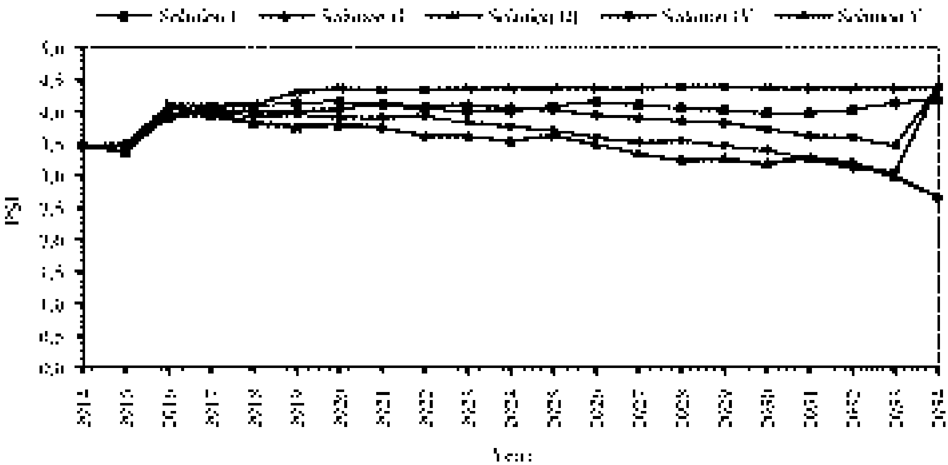

Figure 5 PSI average value for all the road network pavements.

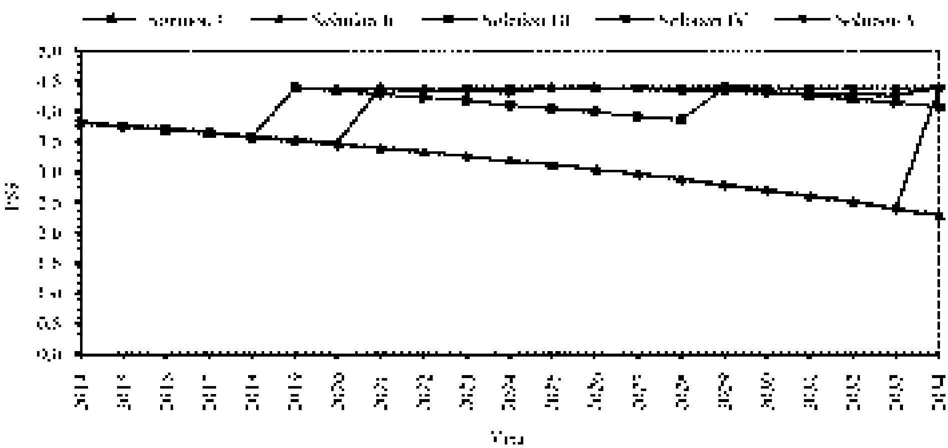

Figure 6 Evolution of PSI for pavement 05001 of a national road.

Figure 6 shows the predicted evolution of the PSI value over the years for pavement section 05001 of a national road as a consequence of the execution of the M&R plan. For this pavement section, which is in good condition (PSI value of 3.81), if solution I ("Knee Point") of MODAT is adopted, only two M&R operations 2 (non-structural maintenance) will be applied to the pavement section, one in year 2018 and another in year 2028. If solution II of MODAT is adopted, no M&R operation will be needed during all the planning time-span. If solution III of MODAT is adopted the recommended M&R operations are very different. The MODAT recommends the application of M&R operation 5 (major rehabilitation) in years 2018, 2022, 2026 and 2030. For solution IV, the MODAT recommends one M&R operation 4 (Medium rehabilitation) in year 2020 and the application of two M&R operation 3 (minor rehabilitation) in years 2024 and 2033. If solution V of MODAT is adopted only one M&R operation will be needed during all the planning time-span, i.e. M&R operation 2 in year 2033. An identical analysis could be made for any other pavement section.

Table 1 Attributes of road sections.

Attributes	Road sections			
Section_ID	05012	05004	05001	05003
Road_class	EN	IC	IP	IC
Pavement_type	Flexible	Flexible	Flexible	Flexible
District	Castelo Branco	Castelo Branco	Castelo Branco	Castelo Branco
Length (m)	21,455	19,439	1931	14,635
Width (m)	5.9	8.8	9.4	8.6
Sub-grade_CBR (%)	5	10	6	4
Structural_number	2.47	3.51	5.20	4.80
Age_of_pavements (years)	16	14	8	3
Annual_average_daily_traffic	744	6,212	4316	5,828
Annual_average_daily_heavy_traffic	100	1000	300	1000
Annual_growth_average_tax	3.0	4.0	3.0	4.0
Truck_factor	2.0	4.0	3.0	4.0
PSI_0	1.79	2.75	3.81	3.90

Table 2 M&R operations to be applied in road sections.

Section	PSI	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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KEY (M&R operations):
1 – Do nothing; 2 – Non-structural maintenance; 3 – Minors rehabilitation; 4 – Medium rehabilitation; 5 – Major rehabilitation.

4 Conclusions

The Multi-objective Decision-Aid Tool (MODAT) is a useful new tool to help the road engineers in their task of maintenance and rehabilitation of pavements. In this MODAT application, the Knee point, which represents the most interesting solution of the Pareto frontier, corresponds to an agency costs weight value of 4%, a user costs weight value of 95% and a weight value of 1% for the residual value of pavements, demonstrating that user costs, which are generally much greater than agency costs and the residual value of pavements, dominate the decision-making process. While the case study of this paper focuses on a national road network, the approach proposed is applicable to any transportation infrastructure network, e.g., municipal road network, bridge network, where the decision-making process often involves multiple objective considerations.

Acknowledgements

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References

[1] Fwa, T., Chan, W., and Hoque, K.: Multiobjective optimization for pavement maintenance programming. Journal of Transportation Engineering, 126 (5), 367-374, 2000.

[2] Wang, F., Zhang, Z., and Machemehl, R.: Decision making problem for managing pavement maintenance and rehabilitation projects. Transportation Research Record, 1853, 21-28, Washington DC, USA, 2003.

[3] Wu, Z., Flintsch, G.: Pavement preservation optimization considering multiple objectives and budget variability. Journal of Transportation Engineering, 135 (5), 305-315, 2009.

[4] Picado-Santos, L., and Ferreira, A.: Contributions to the development of the Portuguese road administration's pavement management system. Proceedings of the Third European Pavement and Asset Management Conference, CD Ed., paper 1138.pdf, 1-10, Coimbra, Portugal, 2008.

[5] Ferreira, A., Picado-Santos, L., Wu, Z. and Flintsch, G.: Selection of pavement performance models for use in the Portuguese PMS. International Journal of Pavement Engineering, 12 (1), 87-97, 2011.

[6] Meneses, S., Ferreira, A. and Collop, A.: Multi-objective decision-aid tool for pavement management, Proceedings of the Institution of Civil Engineers-Transport, 166 (2), 79-94, 2013.

[7] Meneses, S. and Ferreira, A.: Pavement maintenance programming considering two objectives: maintenance costs and user costs, International Journal of Pavement Engineering (ISI Journal), 14 (2), 206-221, 2013.

INFLUENCE OF TIRE PRESSURE ON THE VERTICAL DYNAMIC LOAD APPLIED ON THE PAVEMENT BY A TRUCK'S FRONT SUSPENSION

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Abstract

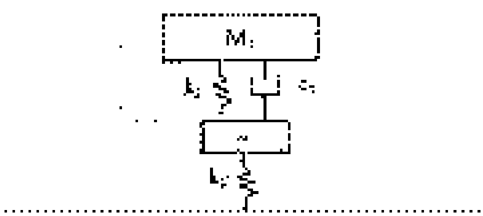
The main objective of this research study is to present the results of the influence of tire pressure, from a truck front suspension, on the vertical dynamic load applied on the pavement. For the measurements, it has been used a durability test track located in Brazil. The tire pressure was increased by 10 psi from 90 to 130 psi with a constant load of 6 tons on the front suspension, the maximum allowed load for front axle, according to Brazilian legislation. By applying relative damage concept, it is possible to conclude that the variation on the tire pressure will not affect significantly the load applied on the pavement. Although, it is recommended to repeat the same methodology, in order to analyse the influence on the variation of the other quarter car model variants.

Keywords: damage, vertical dynamic load, tire

1 Introduction

1.1 Background

Vertical dynamic load is directly related to the deterioration of the pavement [1]. Therefore, this relation can also be extended for the vehicles variants, especially for the commercial vehicles – trucks and buses. By analysing the quarter car model (Figure 1) it was expected that the tire spring rate could influence it by changing the tire pressure.



M_1 sprung mass;
k_1 primary suspension spring rate;
c_1 shock absorber damping force;
m_1 unsprung mass;
k_1' tire spring stiffness.

Figure 1 Quarter car model [2]

1.2 Scope

The main objective of this research study is to analyse the influence of the tire pressure on the vertical dynamic load applied on the pavement.

1.3 Boundaries and assumptions

In order to analyse the influence of the tire pressure, it was necessary to keep constant the other variants of the quarter car model (Figure 1):

- sprung mass and unsprung mass: set as 6 tons, the maximum allowed weight on the front axle according to the Brazilian legislation;
- primary suspension spring rate and shock absorber damping forces: set according to the manufacture specification – new components;
- pavement longitudinal profile: the tests have been performed on a proving ground located in Brazil, in order to keep the same track in all measurements;
- it has been chosen an 8x2 rigid truck (Figure 2) and all measurements reflect the loads of both front steering axles.

Figure 2 Tested truck

2 Methodology

2.1 Instrumentation and calibration

Uniaxial strain gauges were placed on the main leave spring of the 1st and 2nd steering axle (Figure 3) on the left hand side (LHS) and right hand side (RHS) of the vehicle. The recorded values given by the mentioned instrumentation were in με (micro-strain). Therefore, it was necessary to calibrate the system in order to estimate the force applied on the pavement. It has been used a weighting scale and applied different loads on the vehicle body with the objective of having the calibration curves between με and the load applied on the ground in tons (Figure 4). Due to the fact that all tested springs have the same spring rate, all calibration curves have similar characteristics.

Figure 3 Primary suspension spring leaf instrumentation

Figure 4 Weighting scale used for the calibration and example of the spring calibration

2.2 Test procedure

It has been used a pothole track in the proving ground for the measurements, with the following constant conditions: vehicle weight (6 tons per front axle), vehicle speed (40 km/h), original shock absorber setting and original primary suspension spring rate setting. The only variant was the tire pressure: 90 psi; 100 psi; 110 psi (recommended pressure for the applied vehicle weight [3]); 120 psi; and 130 psi – respectively: 621 kPa, 690 kPa, 758 kPa, 827 kPa and 896 kPa. Tire size: 295/80 R22.5.

Figure 5 Detail of the pothole track

3 Results Analysis

3.1 Overview

Figure 6 presents an example of the time signal of each front spring for a tire pressure of 110 psi. The mentioned figure refers to the load applied to the pavement on each front axles tires, in tons. By analysing the time signal data, it is not possible to find a conclusion. Therefore, it was necessary to use others statistical analysis.

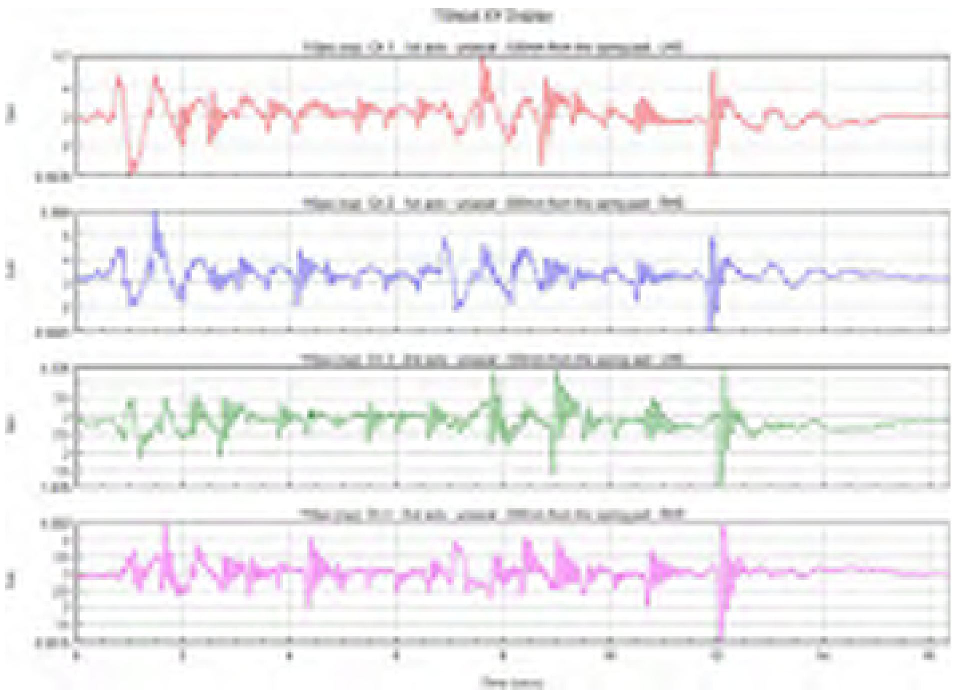


Figure 6 Time signal – load applied on the pavement – 110 psi

3.2 Histograms

Figures 7 to 10 present the histograms for the tested tire pressures. It is possible to visualize on these Figures that the counting cycles close to the static load (3 tons) are the mandatory values. Nevertheless, with this statistical tool, it is not possible to analyse the behaviour of the other loads to the pavement.

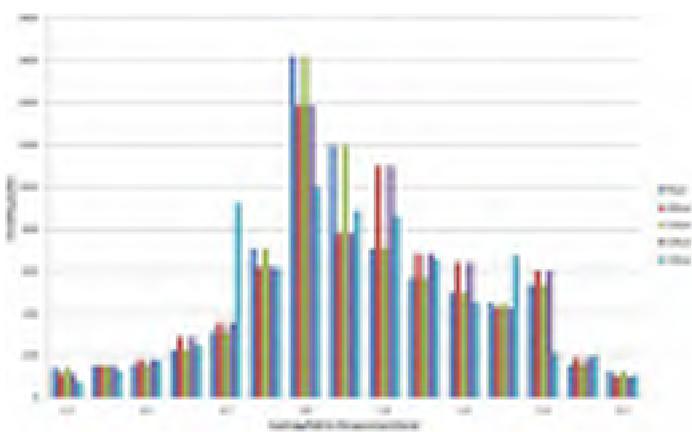


Figure 7 Histogram of the load applied on the pavement – 1st axle LHS

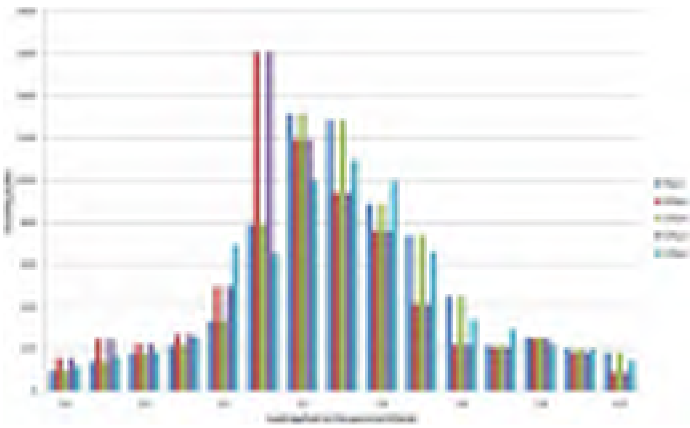


Figure 8 Histogram of the load applied on the pavement – 1st axle RHS

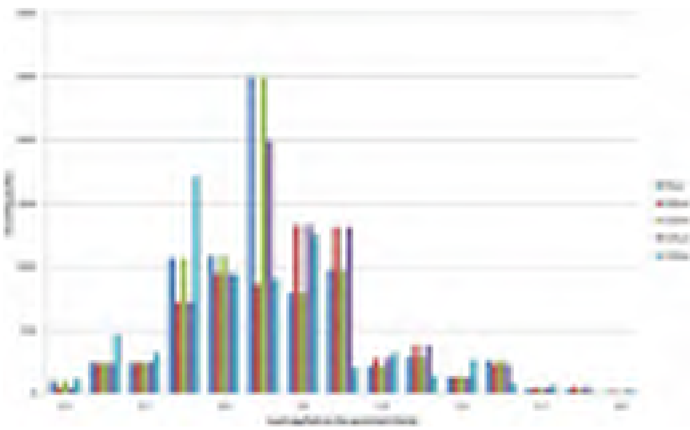


Figure 9 Histogram of the load applied on the pavement – 2nd axle LHS

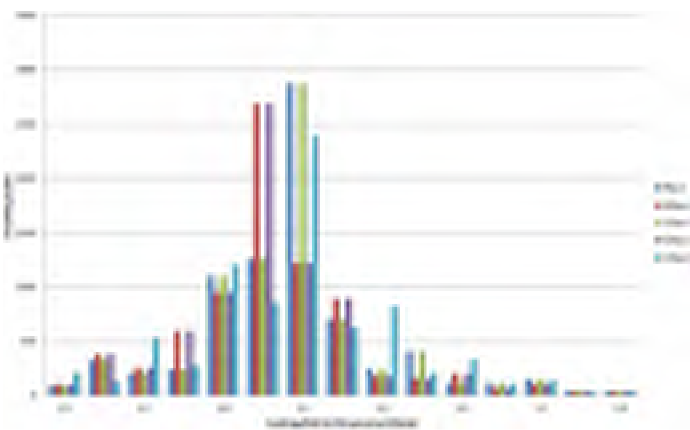


Figure 10 Histogram of the load applied on the pavement – 2nd axle RHS

3.3 Relative damage calculation

Equivalent constant amplitude stress level is calculated for making life estimates for variable amplitude loading, by using rainflow cycle counting (eqns 1 to 3), which is recommended for highly irregular variation of load with time [4].

$$\sigma_a = \frac{(\sigma_{max} - \sigma_{min})}{2} \tag{1}$$

where:

σ_a average of each rainflow cycle (σ_{max}, σ_{min});
σ_{max} max stress for each rainflow cycle counting;
σ_{min} min stress for each rainflow cycle counting;

$$N_{fj} = \frac{1}{2}\left(\frac{\sqrt{\sigma_{max} . \sigma_a}}{\sigma'_f}\right)^{\frac{1}{b}} \tag{2}$$

where:

N_{fj} absolute damage of each rainflow cycle counting;
σ'_f theoretical stress that indicates failure with zero cycles (material property);
b slope of the Stress-Life curve (material property).

$$\sum \frac{N_j}{N_{fj}} : \text{absolute damage} \tag{3}$$

where:

N_j quantity of rainflow cycle counting for each range;

Relative damage will be the ratio between a given tire pressure damage with the baseline (110 psi), Figure 11.

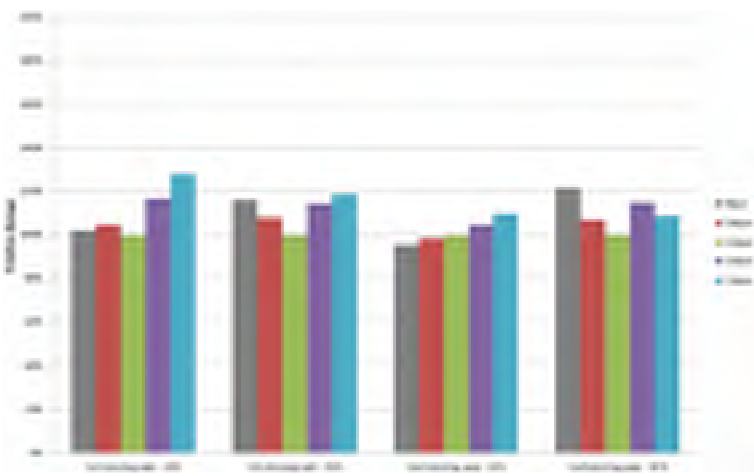


Figure 11 Relative damage among tire pressures, considering k=5

By analysing Figure 11, it is possible to observe that the relative damage, regarding tire pressure, has no significant relation with the vertical load applied on the pavement.

4 Conclusion and recommendations

By applying relative damage concept, it is possible to conclude that the variation on the tire pressure will not affect significantly the load applied on the pavement. Although, t is recommended to repeat the same methodology, in order to analyse the influence on the variation of the other quarter car model variants.

References

[1] Sayers, M. W.: Gillespie, T. D. & Queiroz, C. A. V.: The International Road Roughness Experiment: establishing correlation and a calibration standard for measurements. Washington D. C: The World Bank, pp. 1-453, 1986.

[2] Gillespie, T.: Fundamentals of vehicle dynamics. Warrendale: Society of Automotive Engineers, Inc., pp. 1-495, 1992.

[3] ALAPA. Latin American Association for Tires and Rings. Technical Regulation. São Paulo, pp. 1-351, 2013.

[4] Dowling, N. E.: Mechanical behaviour of materials: engineering method for deformation, fracture and fatigue. 4. ed. New Jersey: Pearson, pp. 1-936, 2012.

DESIGN MODEL FOR STATIC AND IMPACT LOAD AFFECTED PAVEMENTS

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Abstract

Pavements of aerodromes, container terminals, logistic terminals, areas of storage, parking lots, areas of waste utilization and etc. are affected by static and impact loads. These loads strongly influence pavement performance by causing permanent deformations and distresses in the surface and even sometimes pavement failure in the beginning of pavement exploitation. The types of structure, materials and layers thickness are the main factors relative to pavement performance. This paper aims to create a design model for static and impact load affected pavements.

Keywords: static loading, impact loading, pavement distress, pavement structure, design model.

1 Introduction

There are many methods, procedures and guides for pavement structure design for roads or streets where usually is considered a moving heavy vehicle traffic load. There is an attitude that designed pavement is one in which there is no structural deterioration of the pavement either over time of design [1–3]. Mostly pavements are designed for 20 years according to equivalent single axle load which is accepted as 8 t, 10 t, 11.5 t or other depending on dominated traffic loads. However, pavement design procedures up for roads cannot be applied for others special objects such as parking aprons of aircraft, ports and containers terminals, logistics terminals, industrial areas, parking lots, waste recycling areas and etc., where different types of loading – static and impact loads – affect pavements.
Researchers describe wheel load as static, quasi-static or dynamic. A static load represents a constant load, which is defined by geometry of contact and mass of the load. A quasi-static load – a moving constant load, which is defined by stiffness and mass of the load without damping force in the equation of equilibrium. And a dynamic load – a changeable moving load, which involves inertia, damping, stiffness and mass terms in the equation of motion [4, 5]. From experience observed that falling object or special machineries induce impact loads. Impact load represents a force of potential energy, which is defined by multiplying object weight, drop height and acceleration of gravity. These kinds of loading produce many distresses in pavements of special objects.
Consequently, it is very important to design a stable pavement, i.e. pavement structure resistant to static and impact load. In other countries this kind of pavements are designed by using special design charts, guides, manuals and computer programs. However, in Lithuania is neither solution for design such pavements (there is regulated only design for roads (streets) pavements). Contribution to this is a lack of information about influence of different loading type to pavement structure and its performance. The aim of this article is to study types of pavement distresses, to analyze load specification and influence of climatic conditions to pavement performance and to suggest a design model for pavements affected by static and impact load.

2 Distresses of pavement structures affected by static and impact load

In general, pavement degradation is a function of pavement roughness, which is a sum of all types of distresses and increases with pavement age [6–8]. The type of pavement distress depends on the pavement structure and materials properties.
Due to easy construction and rehabilitation flexible pavements are the most common type of pavement structure using in Lithuania and in other countries. However, flexible pavement design life is shorter than rigid pavements and usually after 20 year of exploitation need full rehabilitation. Pavement distresses are classified in to: surface defects (raveling, bleeding and polishing), surface deformations (rutting, shoving, corrugation, bumps, heaves, settlements and blow ups (curling)), cracks (fatigue, thermal, longitudinal and slippage) and potholes. Though, in the asphalt pavements the most common distresses are fatigue cracking, rutting (permanent deformations) and thermal cracking, meanwhile in pavements affected by static and impact load – permanent deformations and thermal cracking.
Thermal cracking is primarily associated with cold temperatures (below 10 °C) at the surface of asphalt layer. Due to low temperature the binder film to get thinner around aggregates. When the temperature drops below the point where asphalt binder becomes brittle, thermal cracking is initiated in the surface of asphalt pavement and immediately grows down [9], [10].
Permanent deformations are associated with viscous-elastic-plastic properties of asphalt and variance of ambient temperature [3, 11, 12]. Four types of permanent deformation may occur in asphalt pavement [13, 14]: surface wear rutting, initial densification, structural deformations and plastic (flow) deformations (rutting). Bitumen binder properties decreases at high temperatures (to +60°C and more) influencing relaxation of asphalt mixture strength [15]. Examples of permanent deformation observe in pavements of special objects are shown in Figure 1. In most cases this kind of deformation develops in surface (wearing) layer of asphalt pavement.

Figure 1 Examples of permanent deformation observe in pavements of special objects: a) port; b) parking lot; c) parking apron of aircraft; d) e) f) industrial area

3 Load specification and climatic conditions

The static and impact load usually delivers from aircrafts, containers, handling equipment (rubber tire gantry cranes (RTGs), straddle carriers, reach stackers, front lift trucks, side loader lift trucks and etc.), storage goods, trucks, cars and/or falling stuff. In areas where is considerable expectation of this kind of loading, the magnitude of loads and loading time is higher comparing with traffic loads. Moreover, loads are concentrated and affect pavement within small contact area. Consequently, it leads high pressure into pavement. Contact pressure strongly depends on contact area load, which is influenced by high of stacking (if source of load is stackable). Into Table 1 is taken contact pressure considering to characteristics of the load source. It is emphasized that contact pressure to pavements affected by static and impact load is significantly higher comparing with contact pressure applied in roads pavements. On purpose to simplify calculations a tire-pavement contact area is assumed as a circle area equals to ellipse's (rectangle) area. This contact area changes by increasing/decreasing magnitude of load and/or by changing inflation pressure [16]. However, common the tire-pavement contact pressure is assumed equal to inflation pressure. Impact load is characterized by drop height and/or weight of falling object. As these characteristics increase, the negative effect of this kind of load to pavement performance increases, too.

Table 1 Contact pressure considering to characteristics of load source[a)]

Object (load source)	Contact area [mm^2]	Maximum high of stacking	Contact pressure [MPa]
Aircraft	[b)]	–	0.3-1.7[c)]
Handling equipment of ports	[b)]	–	0.7-1.7
Container	26250	6-8 (12)[d)]	2.59-12.5[e)]
Heavy vehicle	[b)]	–	0.6-1.0
Trailer dolly (wheel)	8800	–	35-40
Trailer pivot plate	33750	–	2.0
Handling equipment of industrial areas	[b)]	–	0.5-1.0
EuroPallet	116000	2-4[f)]	0.03-0.44[g)]
Car	[b)]	–	0.20-0.25

a) Sources of impact load are not included in to table due to their large variety.
b) The leg of load source is wheel. Contact area of tire-pavement depends on characteristics of tyre.
c) Contact pressure depends on type of aircraft.
d) Fully loaded containers are usually stacked up to 6 high and empty containers – 8 high. In some cases containers can be stacked up to 12 high.
e) When only 1 container is on the pavement the contact pressure is 2.59 MPa, when conteiners are stacked up to 8 high – 12.5 MPa.
f) EuroPallets are usally stacked up to 2 high for transportation and up to 4 high for storage.
g) When only 1 EuroPallet with goods is on the pavement the contact pressure is from 0.03 MPa depending on the weight of goods, when EuroPallets with goods are stacked up to 4 high – by 0.44 MPa depending on the weight of goods.

In design process of special objects should be paid attention to different loading, i.e. static (containers, standing equipment, storage goods and etc.) and slow dynamic loading (handling equipment's movements, cornering, accelerating and braking). Mostly, slow dynamic loading is assessed by applying a load multiplication factor.
Climatic conditions such as temperature, moisture, frost and number of freeze-thaw cycles strongly influence pavement performance and exploitation. Climatic conditions in Lithuania are similar to other cold regions countries. There temperature of asphalt pavement surface varieties from -22°C to +53°C [17]. Moreover, number of freezing/thawing cycles of pavement

surface exceeds 80 times and more in year [18]. Consequently, pavement structure is designed evaluating resistance to frost heave.

It is well known, that moisture influences the behavior of unbound layers, while temperature – the stiffness (elastic modulus) of bituminous layers and thermal stresses in cement layers. Pavement failures (distresses) related with high moisture content in unbound layers is common phenomenon in cold region pavements [19-21]. Increasing water content leads more permanent deformations in unbound layers [20]. Moisture content and pavement stiffness depends on season, because pore water crystallizes into ice lenses during cold season conditioning increase of stiffness of unbound layers [20, 21]. The negative influence of moisture and frost to pavement structure performance is solved by ensuring sufficient drainage and thickness of whole pavement structure, respectively.

Variations of temperature affect the viscous-elastic-plastic properties of asphalt pavement. The elastic modulus decreases as the temperature increases and increases as the temperature decreases [21]. In design process, it is assessed by including the non-linear asphalt behavior.

In the rigid pavements, daily temperature variations lead curling of the slab: the slab blows up (expansion of the pavement surface) when the top of the slab is warmer than the bottom of the slab (daytime) and the slab blows down (contraction of the pavement surface) when the top of the slab is cooler than the bottom of the slab (nighttime). Furthermore, the temperature variations give thermal stresses in the slab, which must be estimated in process of pavement design [22].

4 Pavement structure design model

All types of pavement structure are used for areas affected by static and impact load. According to pavement structure layers and materials mechanical performance pavement structures are:

- Flexible – asphalt layer(-s) or stone/concrete blocks on unbound granular base or asphalt base layer.
- Rigid – concrete layer(-s) or stone/concrete blocks on bound granular base or asphalt base layer or concrete layer.
- Composite – semi-rigid/semi-flexible – ultra-thin concrete layer or concrete paving flags on flexible pavement structure, asphalt layer(-s) on concrete pavement, pours asphalt with cement grout on flexible pavement structure.

Foreign countries practice has shown that traditionally pavements of special objects have been design using design charts. According to them the whole thickness of pavement structure and/or each thickness of pavement layer are selected. However, this design method has low accuracy. Consequently, other design model should be applied instead of design charts. In our opinion design charts could be used only for selection of whole thickness of pavement structure and the upper part of the pavement (generally wearing layer) should be analyzed separately according to development of permanent deformations and thermal cracks.

Considering to foreign countries practice and normative requirements or guides of pavement design for special objects, there is suggested a design model for pavements affected by static and impact load (Fig. 2). A design model consists of three main parts: field of application, input data and analysis procedure. Following design process step-by-step, the designer selects the best solution for new or rehabilitated pavement structure depending on environmental/climate conditions, structural conditions, design materials and loading conditions.

FIELD OF APPLICATION

Parking aprons of aircraft	Ports and container terminals	Logistics terminals	Industrial areas	Parking lots	Areas affected by impact load

INPUT DATA

	Characteristic	For new construction	For rehabilitation
Environmental /Climate conditions	• Environmental temperature	+	+
	• Pavement temperature		+
	• Moisture	+	+
	• Hydrothermal conditions of pavement structure		+
	• Frost heave		+
	• Depth of freeze	+	+
	• Ground water level	+	+
Structural conditions	• Pavement structure composition, condition and materials		+
	• Bearing capacity of pavement structure and separate layers		+
	• Subgrade conditions, materials (soils) and bearing capacity	+	+
	• Embankment conditions and materials (soils)	+	+
	• Distress types and levels		+
Design materials	• Asphalt mixture	+	+
	• Asphalt with cement grout	+	+
	• Portland cement concrete (PCC)	+	+
	• Concrete paving blocks, flags	+	+
	• Layer bonding materials	+	+
	• Bonded granular layer materials	+	+
	• Unbounded granular layer materials	+	+
	• Soils (subgrade/embankment)	+	+
	• Special materials	+	+
	• Special rehabilitation materials		+
Loading conditions	• Type of loading	+	+
	• Gross weight	+	+
	• Contact load	+	+
	• Load interval frequency	+	+
	• Load-pavement contact area	+	+
	• Contact pressure	+	+
	• Maximum drop weight (impact loading)	+	+
	• Maximum drop height (impact loading)	+	+
	• Fatigue loading (history)	+	+

ANALYSIS PROCEDURE

	Model	Criteria	For flexible/ composite pavement	For rigid pavement
Pavement response to load	• Multi-layer linear elastic model	Calculate stresses, strains, deflection	+	+
	• Multi-layer viscoelastic model		+	
	• Finite difference method		+	
	• Boundary element method		+	
	• Finite element model		+	+
Pavement response to frost heave	Empirical (from experience)	• Max. freeze depth	+	+
		• Min. pavement structure thickness	+	+
		• Allowable moisture content in structure (during thaw season)	+	+
Performance (Design) criteria	• Alligator cracking (bottom-up)	• 10-35 % lane area	+	
	• Fatigue cracking (bottom-up)	• 10-45 % lane area	+	+
	• Permanent deformation	• 10-17 mm rut depth	+	
	• Thermal cracking	• 95-133 m/km	+	+
	• Flow deformation	• 10-17 mm rut depth	+	
	• Mean joint faulting	• 3-6 mm		+
	• Slab cracking (frost heave)	• 10-20 %		+
	• Punchouts			

Figure 2 Flow chart of pavement structure design model

5 Conclusions

In Lithuania pavement structures for special purposes are designed according to experience and standardized pavement structure catalogue for roads. There is lack of procedures, requirements, guides and/or manuals for it.
For special purposes pavement structures long-term static or short-term impact loading are assumed as most common negative factor for pavement distress. Moreover due to brittle of bitumen at low temperature thermal cracking is assumed as very significant distress, too.
Considering to specification of static and impact loading, especially to wide range of contact pressure (from 0.03 MPa up to 40 MPa), design procedures for pavement structure of roads and streets cannot be applied for pavements of special objects – terminals, aprons, parking and storage lots.
The negative influence of moisture and frost to pavement structure performance is solved by insuring sufficient drainage and thickness of whole pavement structure, respectively. While influence of temperature is assessed by different stiffness (elastic) modulus of bituminous layers and thermal stresses in cement layers.
Research represents pavement structure design model with three main objectives: field of application, input data (environmental/climate conditions, structural conditions, design materials and loading conditions) and analysis procedure (pavement response to load, pavement response to frost heave and performance (design) criteria).
The established model will be extended after pavement structure materials experimental research results analysis.

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References

[1] LAKD: Design rules of Road Standardized Pavement Structures, Vilnius, Lithuania, pp. 42, 2008.

[2] COST 333: Development of New Bituminous Pavement Design Method, Brussels, Luxembourg, pp. 375, 1999.

[3] Ferne, B.: Long-life pavements – a European study by ELLPAG, International Journal of Pavement Engineering, Vol. 7, No. 2, pp. 91–100, 2006.

[4] Woodrooffe, J. H. F. & LeBlanc, P. A.: Heavy Vehicle Axle Dynamics, International Symposium on Heavy Vehicles Weights and Dimensions, Kelowna, pp. 25–35, 1986.

[5] Organisatio for Economic Co-operation and development: Dynamic Loading of Pavements: Report, Road Transport Research Programme, pp. 184. 1992.

[6] Žiliūtė, L., Laurinavičius, A. & Vaitkus, A.: The Defects of Asphalt Pavements and Their Impact to Draiving Quality in Vilnius City Streets, The 7th International Conference Environmental Engineering, Vilnius, pp. 6, 2008

[7] Fwa, T. F., Pasindu, H. R. & Ong, G. P.: Critical Rut Depth for Pavement Maintenance Based on Vehicle Skidding and Hydroplaning Consideration, Journal of Transportation Engineering, Vol. 138, No. 4, pp. 423–429, 2012.

[8] Mogawer, W. S., Austerman, A. J., Daniel, J. S., Zhou, F. & Bennert, T.: Evaluation of the effects of hot mix asphalt density on mixture fatigue performance, rutting performance and MEPDG distress predictions, International Journal of Pavement Engineering, Vol. 12, No. 2, pp. 161–175, 2011.

[9] Boutin G. & Lupien, C.: Thermal Cracking of Asphalt Pavement, 2nd Eurasphalt and Eurobitume Congress, Barcelona, pp. 45–59, 2000.

[10] Doré, G. & Zubeck, H. K.: Cold Regions Pavement Engineering, pp. 460, 2009

[11] Wasage, T., Statsna, J. & Zanzotto, L.: Repeated Loading and Unloading Tests of Asphalt Binders and Mixes, Road Materials and Pavement Design, Vol. 11, No. 3, pp. 725–744, 2010.

[12] Kim, Y. R., Lee, Y. C. & Lee, H. J.: Correspondence Principle for Characterization of Asphalt Concrete, Journal of Materials in Civil Engineering, Vol. 7, No. 1, pp. 59–68, 1993.

[13] Morosiuk, G. & Riley, M.: Modelling Road Deterioration and Works Effects in HDM-4, Highway development & Management, World Road Association, pp. 296, 2004.

[14] NCHRP 1-37A: Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures, Champaign, Illinois, USA, 2004.

[15] Vaitkus A. & Paliukaitė, M.: Evaluation of Time Loading Influence on Asphalt Pavement Rutting, Procedia Engineering, Vol. 57, pp. 1205–1212, 2013.

[16] COST 334: Effects of Wide Single Tyres and Dual Tyres, Belgium, pp. 170, 2001

[17] Vaitkus, A., Laurinavičius, A., Oginskas, R., Motiejūnas, A., Paliukaitė, M. & Barvidienė, O.: The Road of Experimental Pavement Structures: Experience of Five Years Operation, The Baltic Journal of Road and Bridge Engineering, Vol. 7, No. 3, pp. 220–227, 2012.

[18] Laurinavičius, A., Juknevičiūtė-Žilinskienė, L.: Eleven Years of RWIS Operation in Lithuania: Possibilities for the Use of the Data Collected, The 8th International Conference Environmental Engineering, Vilnius, pp. 1108-1112, 2011

[19] Saevarsdottir, T. & Erlingsson, S.: Water Impact on the Behaviour of Flexible Pavement Structures in an Accelerated Test, Road Materials and Pavement Design, Vol. 14, No. 2, pp. 256–277, 2013.

[20] Simonsen E. & Isacsson, U.: Thaw weakening of pavement structures in cold regions, Cold Regions Science and Technology, Vol. 29, No. 2, pp. 135–151, 1999.

[21] Motiejūnas, A., Paliukaitė, M., Vaitkus, A., Čygas, D. & Laurinavičius A.: Research on the Dependence of Asphalt Pavement Stiffness upon the Temperature of Pavement Layers, The Baltic Journal of Road and Bridge Engineering, Vol. 5, No. 1, pp. 50-54, 2010.

[22] Westergaard, H. M.: Analysis of stresses in concrete pavement due to variations of temperature, Highway Research Board Proceedings, Vol. 6, pp. 201–215, 1926.

ALTERNATIVE REHABILITATION METHODS FOR LOW-VOLUME ROADS

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Abstract

This paper aims to illustrate the behaviour of two technologies – soft asphalt pavement and Otta seal in Lithuania. Measurements of roughness and condition of soft asphalt pavement and Otta seal were carried out in spring after the first winter and after one year of exploitation. Based on the results of this research the conclusions and recommendations were made for further soft asphalt and Otta seal application in Lithuania.

Keywords: low-volume roads, gravel pavement, soft asphalt, soft bitumen, Otta seal, visual assessment

1 Introduction

In Lithuania more than third (34.2 %) of the state roads are gravel roads [1]. Average annual daily traffic (AADT) varies from 40 vehicles per day (vpd) to 1000 vpd on these roads, so often they are called low-volume roads [2]. The main function of low-volume roads is to ensure access to objects i.e., join small towns, farmsteads and other objects with higher-significance roads. Though, the maintenance of these roads needs large budget and operation of them awake social discomfort.

The main problem of maintenance of roads with gravel pavement is dustiness. Dust generation from gravel roads decrease visibility, which leads to reduction of traffic safety and speed. Thus, maintenance costs of vehicles, medical costs of accident victims and costs deaths increases [3].

Chemical dust suppressants are used to prevent the loss of fines, e.g. calcium or magnesium chloride, calcium lignosulphonate and etc. The dustiness can be reduced up to 80 % using these suppressants [4]. However, the efficiency of dust suppressants depends on climatic conditions and rainy days [5]. The use of such dust suppressants is not enough efficient due to wet and rainy climate in Lithuania, i.e., the costs for reduction of dustiness are inadequate for desired performance [6].

In Lithuania it has always been an issue in finding durable and economical solutions for paving low-volume roads, which AADT is up to 500 vpd. Recent 20 years conventional asphalt concrete pavement structures were constructed using paving grade bitumen with a penetration 70/100 or 100/150. The performance of these pavements was satisfactory. After recent winters in Lithuania, when the total freezing index (FI) was high, it was observed that in spring on the low-volume roads with asphalt pavement the extent of frost heaves has increased and more pavement defects have occurred. In order to avoid frost heaves a special attention should be paid to the 30 cm thick upper subgrade which should be laid from less frost-sensitive soils or it is necessary to carry out the soil improvement or strengthening. Those additional works would significantly increase road construction costs (up to 25 %). Over the last decade

on the low-volume roads of Nordic countries the soft asphalt has been used which is less sensitive to frost heaves and fatigue, more flexible and has the ability of self-healing.
As for all roads, the pavement strength must be adequate to carry the anticipated traffic load. One type of surfacing that can provide an economic and practical alternative to traditional surfacing is the Otta seal [7]. The Otta seal design solution does not add strength to the existing gravel surface, but this type of the surface dressing improves the gravel road operational characteristics.

2 Experience of the use of soft asphalt and Otta seal

The largest experience of using soft asphalt has been represented by Finland, Sweden and Germany. Soft asphalt is commonly made of mineral material mixture, filler aggregate and binder – soft bitumen. The soft asphalt mixture uses bitumen of high penetration from 250/330 to 650/900 or soft bitumen from V1500 to V12000. The Swedish General Technical Construction Specification for Roads Road 94 indicates that on roads with AADT ≤ 500 vpd the wearing course can be laid on gravel pavement. However, this type of pavement structure shall meet the requirements to layer thicknesses.
Safwat determined that after 7 years of road service life the extent of pavement distresses was inconsiderable and cold-mix asphalt (mix of 83 % of used asphalt granules, 14 % of new mineral materials and 3 % of new bitumen emulsion produced on a basis of soft bitumen) mixtures can be an alternative to hot-mix asphalt mixtures [8]. Jacobson with the co-authors determined that mechanical properties of soft asphalt may vary depending on its service life, composition and production process [9]. Viscosity of the regenerated bitumen binder may vary from 12000 mm^2/s (just after laying asphalt mixture) to 33000 mm^2/s (after three service years). Jacobson with the co-authors states that when defining mechanical properties of soft asphalt it is very important to select properly the testing temperatures depending on a climatic zone where the test soft asphalt mixture is laid [10].
Having studied the technical documents of various European countries the following advantages of soft asphalt are distinguished: elastic, durable, self-healing, low temperature mixing, well workable, and recyclable [11–13]. Also, soft asphalt has some disadvantages: not stable enough, it could form water film on new constructed asphalt pavement, low light reflection, flux used for binder is environmentally unfriendly, limited resistance to abrasion.
The European standard EN 13108-3 specifies requirements for soft asphalt used on the roads and other trafficked areas. According to the standard EN 13108-3 there are several types of soft asphalt (type A; type B; type C; type S). As the aim of further research was to use as softer and reasonable bitumen as possible, it was selected soft asphalt of the types C and S.
The Otta seal is a particular type of bituminous surface treatment which was originally developed by the Norwegian Road Research Laboratory (NRRL) in the early 1960s. The design of the Otta seal is relatively simple. It relies on an empirical approach that is based on experience in the selection of both an appropriate type of binder and an aggregate application rate [7, 14]. According Overby and Pinard the choice of a particular type of Otta seal in relation to traffic level is broadly [7, 14]. It is recommended to use the single Otta seal with sand cover seal when the AADT < 500 vpd and the double Otta seal when the AADT > 500 vpd.
One of the main advantages of an Otta seal is the flexibility it offers in terms of the variety of materials that can be used for producing the graded aggregate [15]. Both crushed or uncrushed material and a mixture of both can be used for producing graded aggregate for Otta seals [7, 16].
The type of binder used in an Otta seal significantly affects its constructability, durability and ultimate performance. It is therefore critically important that a correct choice of binder is made to ensure that the critical function of a complete coating of the mineral aggregate particles, even the fines, is achieved during construction [14].

3 Experimental research of behaviour of soft asphalt and Otta seal in Lithuania

In 2012 the trial sections were built to clarify the effectiveness and functionality of soft asphalt and Otta seal technologies under current traffic and climate conditions. 26 trial sections with double Otta seal and 5 with soft asphalt were built in 5 regions of Lithuania (Table 1). The trial road sections were selected taking into account according to different influencing factors: estimated design load, AADT (Average Annual Daily Traffic), climatic conditions (freezing index, frost depth), raw materials and contractors.

Table 1 List of trial road sections

Region	Road No.	Road section milestones [km]	AADT [v/d]	AADT (Heavy vehicles) [%]
Trial road section with double otta seal				
Klaipėda-Tauragė	1708	9.70–10.35	76	13.16
	1717	9.19–9.30	231	9.09
	3702	8.92–9.40	76	6.58
	4215	6.40–7.20	157	10.83
	4516 (1)	9.95–10.90	120	8.33
	4516 (2)	10.90–11.53	54	7.41
Šiauliai-Telšiai	1018	5.30–5.83	105	0.95
	2707	0.00–0.60	148	3.38
	2735	2.00–2.62	137	9.49
	3208	17.75–19.48	453	44.15
	4118	9.30–10.00	183	4.92
Panevėžys-Utena	1235	0.80–2.12	138	10.87
	1401	25.40–26.15	134	10.45
	2427	1.70–3.10	100	10.00
	2816 (1)	0.00–0.40	173	2.89
	2816 (2)	2.20–4.24	173	2.89
	3610	2.97–3.63	133	9.02
Vilnius-Alytus	3918	3.43–4.03	190	6.84
	4404	15.64–16.17	231	2.60
	4726	0.00–1.05	278	9.71
	5017	7.52–8.38	205	20.49
Kaunas-Marijampolė	2623	2.60–3.58	114	21.05
	2635	2.20–3.40	194	5.15
	2642	0.00–0.95	95	5.26
	3539	0.00–1.40	137	8.96
	5123	2.50–3.50	85	14.12
Trial road section with soft asphalt pavement				
Kaunas-Marijampolė	1716	3.80–5.20	67	10.45
Panevėžys-Utena	2430	0.65–1.75	179	4.47
Šiauliai-Telšiai	4028	1.11–2.12	652	10.12
Vilnius-Alytus	2518	0.75–3.30	153	3.27
	5235	4.23–5.56	133	11.28

When designing soft asphalt pavement structure, the reference was made to “the General Technical Specifications for Roads – Chapter 3 – Pavement Design”, issued by the Swedish

National Road Administration. The pavement structure was adapted to the regional climatic conditions, frost index, soil type and traffic loads. Finally, in a combination with Lithuanian "Technical Specifications for the Standardized Road Pavement Structures", the soft asphalt pavement structure, given in Fig. 1, was derived.
The contractors were free to select soft asphalt mix type and to perform type testing. All contractors selected soft asphalt SA 16-d-V6000 Type C. The mechanical and physical properties of soft asphalt SA 16-d-V6000 Type C laid on trial sections are presented in the Table 2.
The double Otta seal was applied on the new 7–10 cm base course from crushed rock. The existing gravel pavement (≥ 30 cm) was matched up to the frost blanket course if an existing material of gravel pavement met the requirements of frost blanket course (Fig. 1).
Bitumen emulsion used for double Otta seal was produced from the soft bitumen V3000 or V6000 and was applied 60 % of the nominal content of binder. The amount of the spread binder was 1.8 kg/m^2 using soft bitumen V3000 and 2.0 kg/m^2 – V6000, respectively.

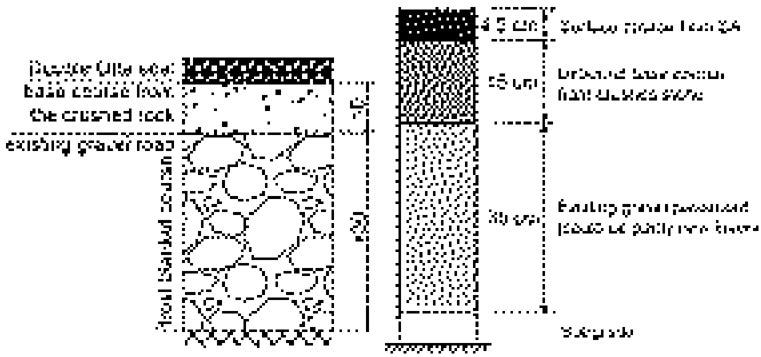

Figure 1 The structure of double Otta seal (on the left) and soft asphalt pavement (on the right)

For double Otta seal 0/16 fraction aggregate was used, which grain size distribution is shown in Fig. 2. The amount (14 l/m^2) of the spread aggregate could be adjusted according to the spread test results. Adhesive additives were not used.

Table 2 The mechanical and physical properties of soft asphalt SA 16-d-V6000 Type C

	Requirements	Road trial section				
		Road No. 1716	Road No. 2430	Road No. 2518	Road No. 4028	Road No. 5235
Bitumen content [%]	$B_{min\ 4.5}$	4.7	5.0	4.7	4.8	4.6
Air voids content V [%]	$V_{min\ 4.0}\ V_{max\ 9.0}$	5.5	6.5	5.5	6.9	6.3
Indirect tensile strength ratio [%]	$ITSR_{60}$	60.0	99.8	66.0	68.0	71.0

Research of functionality of trial sections included evaluation of performance characteristics during different seasons. Performance characteristics were evaluated measuring roughness (International Roughness Index (IRI)) and doing the visual assessment of defects. The measurements of roughness of trial sections with double Otta seal and soft asphalt were made at the beginning of service of trial sections and after 1 year. The visual assessment of defects of trial sections with double Otta seal were performed in autumn, in spring and in autumn (after 1 year), and on the sections with soft asphalt – in spring and in summer.
During spring thaw and summer, after the first winter of the service of trial sections with soft asphalt, a visual assessment of their condition was carried out. The following defects were assessed: longitudinal cracking, potholes, ravelling, seals and bleeding.

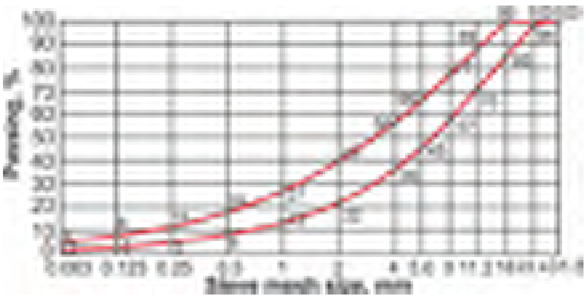

Figure 2 Gradation curves for the double Otta seal aggregate

All defects of trial sections with double Otta seal were grouped into: P_1 – fatting up, bleeding and tracking, %; P_2 – scabbing, tearing and longitudinal joint crack, %; P_3 – corrugation and blister, %; P_4 – streaking, m.

4 Results of experimental research and their analysis

The results of roughness of soft asphalt pavement at the beginning (in November of 2012) of service of trial sections varied from 1.28 m/km to 1.64 m/km and after 1 year (in August of 2013) – from 1.33 m/km to 2.10 m/km (Fig. 3), i.e., roughness increased in average 3 %. The requirement to the IRI of low-volume roads is 3.5 m/km. Also, the results show, that the roughness of pavement is close to the requirements applied to the main heavy duty roads, which is 1.5 m/km.

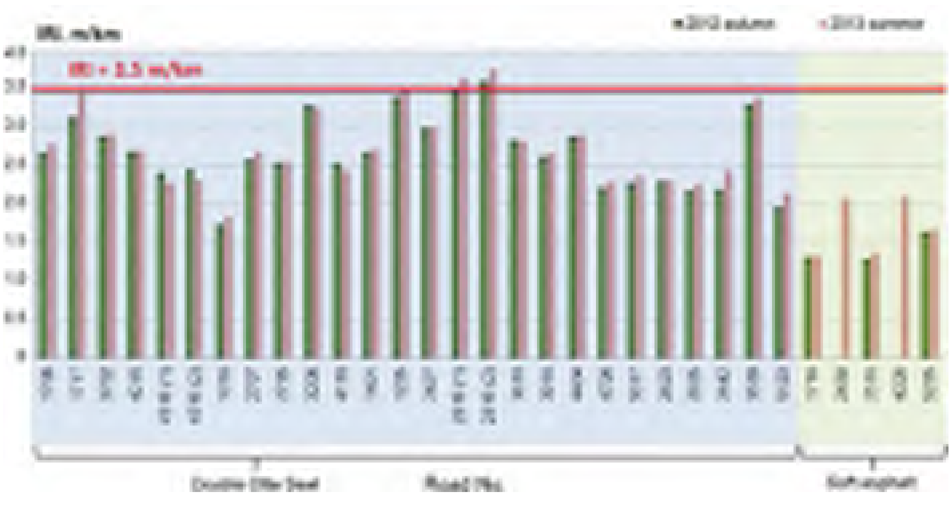

Figure 3 Roughness of trial sections with double Otta seal

The roughness of trial sections with double Otta seal was measured twice as with soft asphalt pavement. The results of roughness are presented in Fig. 3. The roughness increased on most test road sections compering measurements made in November 2012 and in August of 2013. Except several test road sections where the roughness decreased, i.e., IRI decreased from 2.41 m/km to 2.28 m/km on the road Nr. 4516(1) and from 2.45 m/km to 2.31 m/km on the road Nr. 4516(2) in Klaipėda–Tauragė region, and from 2.53 m/km to 2.43 m/km on the road Nr. 4118 in Šiauliai–Telšiai region (Fig. 3).

It was also determined that IRI met the requirement (IRI ‹ 3.5 m/km) on almost all the test sections except road Nr. 2816(1) where measurements were made in August of 2013 and road Nr. 2816(2), where measurements were made in November of 2012 and in August of 2013, i.e., IRI › 3.5 m/km (did not meet the requirement). These two test road sections are in the Panevėžys–Utena region.
The results of visual assessment of trial sections with soft asphalt made in summer shows that longitudinal cracks has decreased due to self-healing ability under high asphalt pavement temperature (Fig. 4). Longitudinal cracks decreased by 62.98 % in road No. 2430, and in road No. 1716 – healed. These results confirm the advantage of soft asphalt, but longitudinal cracks not always heal completely.

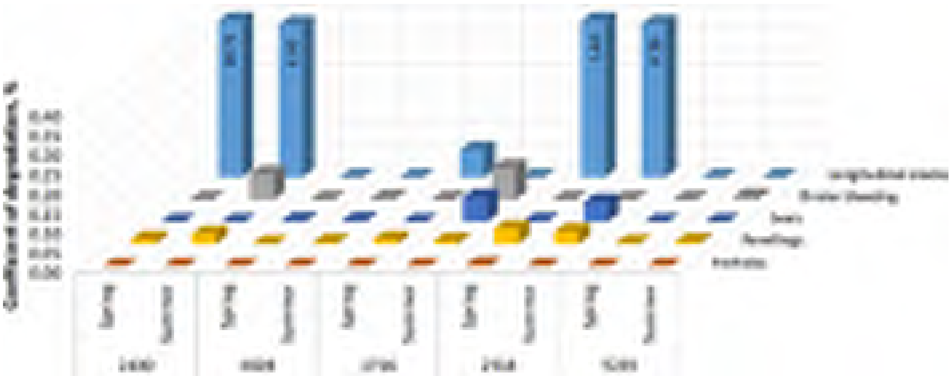

Figure 4 Results of the visual assessment of the trial sections with soft asphalt

During the visual assessment in summer the bleeding of soft asphalt pavement was observed, but the amount of it was small. It doesn't have impact to maintenance of pavement. But, the bled bitumen sticks to tyres of vehicles, and this process could increase ravellings. In roads No. 1716 and No. 2518 the decrease of ravellings was observed, i.e., respectively by 21.18 % and 34.00 %, which could form due to bleeding of bitumen. Though, in roads No. 2430 and No. 5235 it was observed significant increase of ravellings, i.e., respectively by 112.50 % and 400.00 %.
The amount of seals increased 2500 % or 25 times in road No. 2518. The analysis of results showed that main reason is an inadequate composition of soft asphalt mix and/or physic and mechanical properties.
The visual assessment of trial sections with double Otta seal was made in autumn of 2012 (at the beginning of service of trial sections), in spring and in autumn of 2013 according to qualitative visual assessment methodology prepared by authors. Results of the visual assessment of trial sections with double Otta seal are shown in Fig. 5 and Fig. 6.
During the visual assessments it was found that the most significant defects were of P1 group (fatting up, bleeding and tracking). Defects of group P2 (scabbing, tearing and longitudinal joint crack) were found less in autumn of 2013 comparing with results found in spring of 2013 and in autumn of 2012. Defects of group P3 (corrugation and blister) were found less than other groups (P1, P2 and P4). The streaking (defect of group P4) was found in 4 of the 26 road sections, i.e., 145.00 m – in road No. 4726 and 20.00 m – in road No. 4404 in Vilnius–Alytus region, 5.00 m – in road No. 3208 and 156.00 m – in road No. 2735 in Šiauliai–Telšiai region.

Figure 5 Results of the visual assessment of the trial sections with double Otta seal

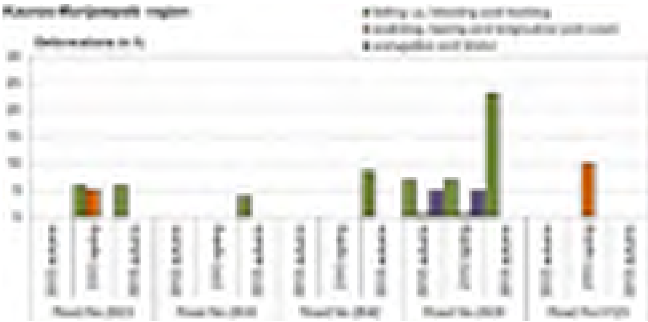

Figure 6 Results of the visual assessment of the trial sections with double Otta seal

5 Conclusions

Based on the experience of foreign countries and short term researches made in Lithuania it was proved that double Otta seal and soft asphalt technologies applied on gravel roads with AADT less than 500 vpd are applicable.
The average roughness of all the trial sections with soft asphalt pavement (from 1.33 m/km to 2.10 m/km) or most of the trial sections with double Otta seal (from 1.84 m/km to 3.76 m/km) complies with the requirements (≤ 3.5 m/km) to the roads of regional significance, and roughness is similar to that of the sections with asphalt base course-pavement.
Trial road sections with soft asphalt pavement are in good condition after 10–12 months of operation (including period of winter and summer). The main defect as longitudinal cracks were observed only in 3 trial sections of 5, and in one of them they healed. Longitudinal cracks decreased by 36 % compering to maximum length of them. Average length of longitudinal cracks varied from 0.7 m to 180 m per 1 kilometre in winter and varied from 0 m to 69 m per kilometre after summer.
Most significant defects (fatting up, bleeding and tracking) were found in the test road sections after one year of operation. The causes were inappropriate aggregate of the double Otta seal and too high binder content and too soft (low viscosity) binder, as well as improperly carried out maintenance of the test road sections in summer. The causes of scabbing, tearing, and longitudinal joint crack, corrugation and blister were unqualified procedure of applying the double Otta seal. The cause of streaking was not enough binder and an uneven binder distribution within the nozzle beam.
The analysis of results showed that there is no correlation between amount of defects and AADT and traffic of heavy vehicle in it.

References

[1] Lithuanian Road Administration under the Ministry of Transport and Communication of the Republic of Lithuania, http://www.lakd.lt/lt.php, 16.12.2013.

[2] Skrinskas, S. & Domatas, A.: Analysis of Lithuanian Gravel Roads Paving Programme Implementation in 1998 – 2005, the Baltic Journal or Road and Bridge Engineering, 1(4), pp. 157–166, 2006.

[3] Monlux, S. & Mitchell, M.: Chloride Stabilization of Unpaved Road Aggregate Surfacing, Journal of the Transportation Research Board, 1989(2), pp. 50–58, 2007.

[4] Edvardsson, K., Gustafsson, A. & Magnusson, R.: Dust Suppressant Efficiency Study: in Situ Measurements of Dust Generation on Gravel Roads, International Journal of Pavement Engineering, 13(1), pp. 11–31, 2012.

[5] Edvardsson, K.: Gravel Roads and Dust Suppression, Road Materials and Pavement Design, 10(3), pp. 439–469, 2009.

[6] Vorobjovas, V.: Assurance of The Function of Low-volume Roads for the Improvement of Driving Conditions, The Baltic Journal of Road and Bridge Engineering, 6(1), pp. 67–75, 2011.

[7] Overby, Ch. & Pinard, M. I.: The Otta Seal Surfacing. A Practical and Economic Alternative to Traditional Bituminous Surface Treatments, Transportation Research Board the 92th Annual Meeting, Washington D.C., 13 p, 2013.

[8] Safwat, S.: Cold Technique of Asphalt Paving, Swedish National Road and Transport Research Institute VTI, 2000.

[9] Jacobson, T., Hakim., H. & Safwat, S.: Undersokning av mekaniska egenskaper hos tunna belaggningar av MJAB, Etapp I [Analysis of Mechanical Properties of Soft Asphalt MJAB, the 1st Stage], Swedish National Road and Transport Research Institute VTI, 2006.

[10] Jacobson, T., Hakim., H. & Safwat, S.: Utvardering av massabelaggning med mjukgjord bitumen, MJAB och MJAG, Etapp II [Evaluation of Properties of Asphalt Mixtures MJAB and MJAG with Modified Bitumen, the 2nd Stage], Swedish National Road and Transport Research Institute VTI, 2006.

[11] Supplementary Technical Contract Specifications and Guidelines for the Construction of Carriageway Surfacing from Asphalt ZTV Asphalt – StB 2000. Roads and Traffic Research Society, Asphalt Roads Working Group. Berlin, 2000. 58 p.

[12] General Technical Construction Specifications for Roads (Chapter 1, Common Prerequisites; Chapter 3, Pavement Design; Chapter 6, Bitumen-bound Layers), Swedish National road administration. 137 p., 1996.

[13] Finnish Asphalt Specifications, Finnish Pavement Technology Advisory Council PANK, 66 p., 1995.

[14] Overby, Ch. & Pinard, M. I.: Otta Seal Surfacing. An Economic and Practical Alternative to Traditional Bituminous Surface Treatments, 37 p., 2007.

[15] Overby, Ch.: Otta Seal – a Durable and Cost-Effective Global Solution for Low-Volume Sealed Roads, the 9th REAAA Conference, Wellington, 1998.

[16] Overby Ch.: A Guide to the Use of Otta Seals, Norwegian Public Roads Administration, 97 p. 1999.

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CONSIDERATION REGARDING ASPHALT MIXTURES IN ROAD PAVEMENT AND AIRPORT PAVEMENT

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Abstract

Asphalt mixtures for airport present certain features comparatively to those for highways, main roads or streets of different technical categories, from composition point of view. Also, laboratory studies include additional specific tests according to European norms in the case of asphalt mixtures for airport. Beside this, the design of an airport pavement is different from those of a road pavement. This paper aims to draw up comparison between the requirements of airport asphalt mixture and road asphalt mixture from laboratory point of view and airport pavement and road pavement from design point of view. It presents laboratory studies and comparative case study.

Keywords: airport, road, design, asphalt concrete

1 Introduction

The airport pavement and road pavement belong to the same structural family. Both are required to build a platform to resist to a given level of traffic and traffic must be done in a safe and comfortable conditions. Due to the specific of airport surfaces, traffic loads are varied, [1-3]. Essential characteristics that differ between road surfaces and airport surfaces are due to applied loads. In the roads case, applied loads shows a slight lateral dispersion (if lane is less than 3.5 m, in alignment) which generate the rutting phenomenon; for airports, the traffic is dispersed (especially on runway) because of diversity of landing train configurations, [2, 3]. Frequency loads is different in case of airport surfaces comparing to road surfaces: thousands (tens of thousands)/day for high-traffic roads and only a few hundred/day for high-traffic airports. Aircraft traffic is significantly different of the road traffic. In the road field, axle load varies from country to country, is around 100 kN, with the pneumatic pressure around 0.8 MPa. In aeronautics, an airplane can transmit more than 900 kN, with a standard air pressure of 1.25 MPa, which can reach to 1.5 – 1.7 MPa.
On the roads, speed is variable depending on the technical class of the road (less than 100 km/h), but on airport surfaces speed is independent of airport/country (continuously variable for runway: more than 300 km/h and constant for taxiways). Peculiarities of those two types of surfaces are given by uniformity (flatness), roughness (adhesion), environmental conditions and deviation, interruption of traffic. In the road field, uniformity (flatness) represents the comfort and safety of passengers, while in aeronautical field the uniformity (flatness) has influence on render safety and the landing – takeoff maneuvers. Flatness has a great influence to the aircrafts damage. Roughness is important both for roads, where the polisaj phenomenon may occur, and for airport runways where tire wear is high and rubber deposits are important. Deviation or interruption of the traffic in case of road interventions is not a huge problem, due to the bypasses, while in the case of airport surfaces is an important issue, [2, 3].

2 Objectives

The objectives of this research are to present similarities and differences between a flexible road pavement and an airport asphalt mixtures, from the recipe, testing laboratory and pavement design point of view. Laboratory studies were done in the Roads Laboratory of the Research Center “Roads and Airports” from Faculty of Railways, Roads and Bridges, Technical University of Civil Engineering Bucharest.

3 Laboratory studies

3.1 Asphalt mixtures recipe

Among the materials component of a flexible pavement the asphalt mixture is considered to be the most important material to be characterized accurately. As is well known asphalt mixture should be as flexible at low temperatures to prevent cracking, and rigid enough at high temperatures to prevent rutting. A good behavior of asphalt mixture in exploitation requires a well designed asphalt mixture recipe and a proper compaction in situ.
Normative in force in our country regarding grading curve for asphalt mixtures for roads is AND 605/2013, [4]. Since Romanian rules do not provide requirements for the design of airport asphalt mixture recipe, to establish aggregate mixture has proposed a grading curve that followed French Design Manual LCPC 2007, [5]. We considered the comparison of two types of asphalt mixtures: BA 16 (asphalt concrete used in wearing course of an road pavement, with 16 mm nominal maximum size) and BBA 16 (asphalt concrete used in wearing course of an airport pavement, with 16 mm nominal maximum size), for highlight the characteristics of the two types of asphalt mixtures (Fig.1) in accordance to EN 13108-1 norm as asphalt concrete used in wearing course, [6].
Restrictions imposed by French Design Manuel LCPC regarding the grading curve are [5]: aggregates below 16 mm size should be set between 90 and 100%; aggregates below 6.3 mm size must be between 65 and 80%; grading curve can be both continuous and discontinuous; less than 2 mm aggregate size should be between 35 and 45%;

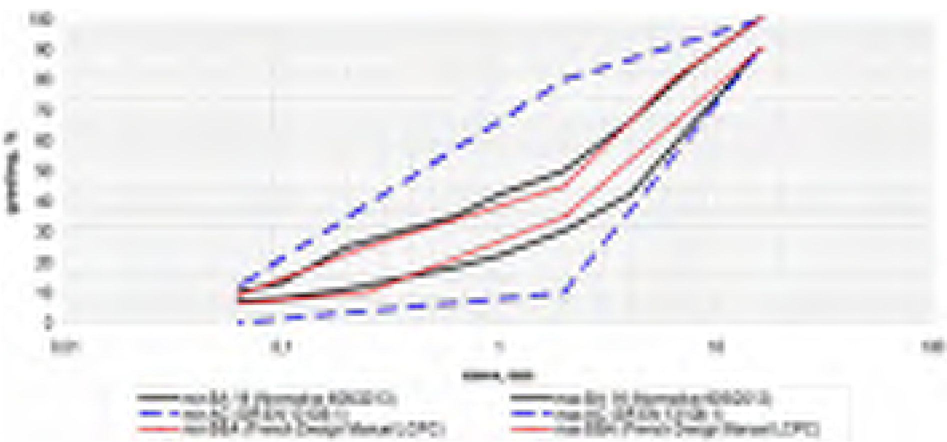

Figure 1 Comparison grading curves

Percent of bitumen was determined based of the method proposed by Duriez (1950):

$$TL = K \cdot \alpha \cdot \sqrt[5]{\Sigma} \qquad (1)$$

where:

K richness modulus (3.75 – 4 for road and 4 – 4.25 for airport);
α correction coefficient relative to the density of aggregates;
Σ specific surface area, expressed in square meters per kilogram, [5].

Restrictions imposed by French Design Manuel LCPC for BBA16 are: the used bitumen can be unmodified or modified; the percentage of bitumen must be above 5.2% ($TL_{min5.2}$ – according to SR EN 13108-1) and restrictions imposed by Normative 605/2013, for BA 16 are: the used bitumen can be unmodified or modified; the percentage of bitumen must be between 5.7 % – 6.5 % ($TL_{min5.6}$ – $TL_{min6.4}$ – according to SR EN 13108-1), [4, 5, 6].
The recipe is designed for an asphalt mixture used in wearing course of an airport pavement; the mixture has 16 mm nominal maximum size.
Knowing this, we further proposed to use two asphalt mixture recipes, one for road, BA 16 (designed according to Romanian standards) and one for airport, BBA 16 (designed according to French Design Manuel), in order to compare their performances taking into account general and fundamental characteristics.
The used materials and recipe of asphalt mixture BBA16 are: aggregates 29% sort 8/16, 23% sort 4/8, 37% sort 0/4; limestone filer 11%; bitumen 5.3% type 45/80 Fr.
The used materials and recipe of asphalt mixture BA16 are: aggregates 25% sort 8/16, 22% sort 4/8, 45% sort 0/4; limestone filer 8%; bitumen 6.1% type 50/70.

3.2 Type of tests

The type testing procedure applied on roads and airports is defined by standards; it has been characterized by an approach based to the greatest extent possible on asphalt mix performance. For structural type materials, it may be classified within the "fundamental" approach. For other material types, the approach is qualified as empirical, as intended in the European standardization, even though it involves "performance related" testing, [3, 5, 6].
Type testing imposes specifications on the components, and especially on the aggregates. It relies upon tests on the gyratory shear press, water resistance, rutting resistance, stiffness modulus and fatigue resistance.
The specific tests have been chosen depending on the type testing level (from 1 to 4, the level 0 doesn't include tests). This type testing level typically depends upon: the type of mixture, the position of the bituminous mixture layer in the pavement, its thickness, projected traffic levels, any special loadings, [5].

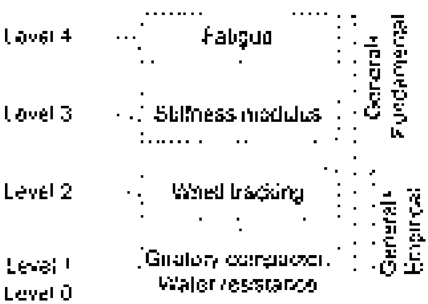

Figure 2 The general, empirical and fundamental approach for asphalt mixtures

According the definitions of EN 13108-1, level 0, level 1 and level 2 are relevant of the general + empirical approach and level 3 and level 4 of the general + fundamental one (Fig.2), [5, 6].

The asphalt mixtures used in the airport area, especially the area of taxiway and the apron must satisfy besides usually requirements for roads, some requirements related to resistance to fuel and de-icing agents according to European norms.
Determination of the water resistance sensitivity according to method A involves dividing a set of cylindrical specimen in two equivalent lots. Determination of the ratio betwen indirect tensile strength for the conditioned lot in water and for the dry lot, expressed as a percentage.
The wheel tracking test involves determining susceptibility of asphalt mixtures subject to deformations, susceptibility evaluated by measuring the rut depth formed by repeated passes of a loaded wheel at a fixed temperature.
Asphalt mixture stiffness is determined by either a complex modulus test (sinusoidal loading on a trapezoidal or parallelepiped specimen) or uniaxial tensile test (on a cylindrical or parallelepiped specimen). The load is applied over a domain of small deformations, through controlling time or frequency, temperature, and the loading law, [5].
Fatigue testing involves determining fatigue life of asphalt mixtures by alternative tests and includes bending tests and direct and indirect tensile tests, under a sinusoidal load or other loads controlled, by using different types of samples and support.
Further are presented the limitations of two types of asphalt mixture considering the presented above tests according to Design Manuel LCPC and AND 605/2013 Norm (Table 1). Test conditions are according to SR EN 13108-20 and the categories of values are according to EN 13108-1, [4-7].

Table 1 Type testing and test conditions for asphalt mixtures BBA 16 and BA 16

Mixture	**Type testing and test conditions according to SR EN 13108-20**			
BBA16	Void content, 80 girations	Laboratory results	Values	5.05
			SR EN 13108-1	V_{min5}
		French Design Manuel LCPC, limits		V_{min3}, V_{max7}
BA16		Laboratory results	Values	3.7
			SR EN 13108-1	$V_{min3.5}$
		Normative 605/2013, limits, max.		V_{max5}
BBA16	Wheel tracking 60°C, small size device procedure B, conditioning in air	Laboratory results	Values	4.88
			SR EN 13108-1	PRD_{AIR5}
		French Design Manuel LCPC, large size device, limits		$P_{7.5}$
BA16		Laboratory results	Values	17.68
			SR EN 13108-1	$PRD_{AIR\ NR}$
		605/2013 Norm, limits, max.%		PRD_{AIR5}
BBA16	Stiffness, 2PB-TR, 15°C, 10Hz	Laboratory results	Values	9054
			SR EN 13108-1	$S_{min9000}$
		French Design Manuel LCPC, limits		$S_{min7000}$
BA16	Stiffness, IT-CY, 20°C, 124μs	Laboratory results	Values	10243
			SR EN 13108-1	$S_{min9000}$
		Normative 605/2013, limits, min.		4600
BBA16	Resistance to fatigue, 2PB-TR, 10°C, 25Hz	Laboratory results	Values	$\varepsilon_6 = 230$
			SR EN 13108-1	ε_{6-220}
		French Design Manuel LCPC, limits		$\varepsilon_6 = 130$

4 Design flexible structure

4.1 Road

Flexible pavement design for roads is carried out in our country according to PD 177 Norm and assume following steps, [8]:
- establishing the traffic corresponding to a perspective period, expressed in 115KN standard axles, equivalent to vehicles which will travel on the road;
- establishing the bearing capacity at the formation level, depending on the type of soil, the climatic type of the area in which the road is located and hydrological regime of pavement;
- choosing the layers for pavement, consideration the materials prevalent in the region, minimum thickness constructive, maximum thickness, etc.;
- the pavement analysis to the standard axle load, considering the thickness of each layer and deformation characteristics of materials and the foundation soil;
- establishing the pavement behavior under traffic involves comparing the calculated specific strain values with the admissible values. This means compliance with the criterion of specific tensile straine on the based of asphalt layers (ε_r), which involves determining the rate of degradation to fatigue (RDO) as the ratio between calculated traffic (N_c) and the admissible traffic (N_{adm}) and the criterion of specific admissible vertical straine at formation level (ε_z);

To illustrate which is presented above, we considered a road pavement by following: 4 cm asphalt concrete BA 16 used in the wearing course (dynamic modulus = 3600 MPa, Poisson coefficient = 0.35), 6 cm asphalt concrete AB 16 used in the base course (dynamic modulus = 5000 MPa, Poisson coefficient = 0.35), 15 cm crushed stone (a dynamic modulus = 400 MPa, Poisson ratio = 0.27), 15 cm ballast (dynamic modulus = 172 MPa, Poisson coefficient = 0.27) and a type of soil P2 (sand and gravel, a dynamic modulus = 90 MPa, Poisson coefficient = 0.30). The traffic volume to which is loaded the road pavement is Nc = 0.95 m.o.s. (heavy traffic). With ALIZE program (elastic multi-layers) it was determined the specific straine values which we have compared with the admissible values according with Table 2.

Table 2 Verification of the flexible structure for roads

ε_r, microdef	ε_z, microdef	RDO	$\varepsilon_{z,\ admisibil}$, microdef	RDO $_{admisibil}$	N_c (m.o.s)
214.5	708.4	0.70	608.7	0.90	0.95

4.2 Airport

In our country there are no norms for design flexible pavement for airports, but one of the methods which take into account is the French method, which involves the following steps, [2, 9]:
- traffic forecasting during exploitation;
- determination the characteristics of the foundation soil, characterized by CBR (Californian Bearing Ratio);
- reviewing climatic factors;
- calculation of equivalent thickness of road pavement depending on the CBR value of the earth foundation and the adopted calculation load.

Actual thickness of the road pavement is obtained taking into account the constructive thickness and the equivalence coefficients that for treated materials with binders are considering the influence of temperature. The minimum equivalent thicknesses of the binder bounded layers is determined depending on the equivalent thickness of road structures and CBR. In order to compare the method of design for an airport pavement to the method of road pavement design we considered that knowed the load which acts to runways, P'' = 73.2 t, type

of aircraft (B747-200), type of landing gear (bogie), foundation soil characterized by CBR = 10 (soil type P2) and the traffic corresponds to a period of exploitation of 10 years, by a factor of 1/3.65 and 10 overlapping movements / day.
With abacus corresponding for airplane type and CBR's were determined equivalent thickness of the road structure He = 57.25 cm. Considering the relation for the equivalent thickness calculation:

$$H_e = \sum_{i=1}^{n} h_i \cdot c_i \tag{2}$$

and equivalence coefficients resulted the following airport pavement: 6 cm asphalt concrete BBA 16 used in the wearing course (c_i = 2), 10 cm asphalt concrete EME used in the base course (c_i = 1.9), 15 cm crushed stone (c_i = 1), 15 cm ballast (c_i = 0.75).

5 Conclusions

There are no complete similarity between pavement for roads and pavement for airports. Unification of international technical standards for roads is recommended, and in the case of airport surfaces is obligatory (done by norms / recommendations ICAO). Grading curve of asphalt mix designed by French Design Manuel LCPC fall within the limits of grading for asphalt concrete AC 16 and for asphalt concrete BA 16. Characteristics of the designed asphalt mixtures is falling in the limits of EN 13108-1. Limits of values for voids volume to 80 gyrations, wheel tracking and stiffness modulus are near, but the airport asphalt mixture, according with French Design Manuel LCPC must satisfy the conditions of sensitivity to water and fatigue conditions which the Romanian norm does not provide for road asphalt mixture.
It should be noted that both pavements for roads and for airports have the same advanced technique, contrary for the each specifications. It is the natural evolution of design methods, of the materials and methods of construction.

References

[1] Surlea, C.: PhD Thesis: Mixturi bituminoase aeroportuare supuse incarcarilor repetate Bucuresti, Romania, 2011;

[2] Zarojanu, H. & Bulgaru, G.: Aeroporturi, Iasi, Romania, 2010;

[3] Service technique de l'aviation civile: Enrobes hydrocarbones et enduits superficiels pour chaussees aeronautiques, Guide d'application des normes, 2009 ;

[4] Normative AND 605: Mixturi asfaltice executate la cald. Conditii tehnice pentru proiectarea, prepararea si punerea in opera, Bucuresti, Romania, 2013;

[5] Laboratoire central des ponts et chaussees. Reseau Scientifique et Technique de l'Equipement: LCPC Bituminous Mixtures Design Guide, 2007 ;

[6] SR EN 13108-1: Bituminous mixtures.Material specifications. Asphalt concrete, 2007;

[7] SR EN 13108-20: Bituminous mixtures.Material specifications. Type testing, 2011;

[8] PD 177: Normativ pentru dimensionarea sistemelor rutiere supple si semirigide, Bucuresti, Romania, 2001;

[9] L'Instruction Technique sur les Aérodromes Civils. Chapitre 5. Conception des chaussées aéronautiques., 1999 .

IMPACT OF HIGH PROCESS TEMPERATURE ON VISCOELASTIC PROPERTIES OF POLYMER MODIFIED BITUMEN IN WATERPROOFING AND BRIDGE PAVEMENTS

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Abstract

This paper presents the most widespread materials and technological solutions of pavement systems on bridge decks used in countries of central Europe, including Poland, with particular emphasis on the impact of high process temperatures on bridge surfacing systems durability. In Europe, the dominant system solutions on bridges is to lay on the steel or concrete deck bridge waterproofing layer (insulation) and then asphalt layers, consisting of waterproofing protective course and a wearing course. For the bridge pavements several kinds of asphalt mixtures containing polymer modified bitumen can be used. In recent years, the protective and wearing courses are mostly built from mastic asphalt. In the research was proved that the impact of high process temperatures (above 250°C) was particularly destructive for commonly used polymer modified bitumen in bridge pavements and waterproofing layers.

Keywords: bitumen, polymer, waterproofing, bridge pavement

1 Introduction

Properly designed and constructed pavement on a bridge deck should meet the following requirements [1], [2]:

- sealed, resistant to water and de-icing measures;
- stable, resistant to deformation bridge deck;
- durable in diversified operating temperatures to ensure a long service life;
- resistant to thermal cracking and fatigue at low and medium operating temperatures and resistant to rutting at high operating temperatures;
- resistant to shear stress;
- rough, ensuring comfort and traffic safety;
- well bonded with waterproofing and bridge deck;
- lightweight, while maintaining sufficient thickness to provide protection of the bridge deck.

On the bridge decks it is possible to construct pavements in both cement concrete and asphalt technologies. In Poland (and in Europe) asphalt pavements are commonly applied. Bridge concrete pavements are commonly used material and technological solution in the United States but asphalt pavements are rarely used. As one of the reasons for avoiding bridge asphalt pavements in the United States is possibility of water penetration through the conventional asphalt layers that cannot be compacted by vibration on bridge structures [3].
In Europe and in Poland it is common to lay on a previously primed surface the waterproofing layer and afterwards the asphalt pavement courses (Figure 1). Pavement together with waterproofing compose a pavement system. This system is the most common used technological solution on steel and concrete bridges. Current usage of polymer modified bitumen

in pavement systems (waterproofing + pavement) enforces sharp technical requirements not exceeding the temperature limits of technological process, which according to the polymers manufacturers instructions should be in the range of about 200°C. It is permitted to raise the temperature to 220°C temporarily, up to two hours. Numerous failures of asphalt pavement systems containing polymer modified bitumen were caused by non-compliance with the temperature requirements. The layout of asphalt pavements on a bridge deck is shown in Figure 1.

Figure 1 Diagram of the pavement system on a bridge deck [4].

The asphalt pavement laid on a waterproofing layer consists of a protective and wearing course. The protective course protects the waterproofing from damage during constructing of pavement upper layers and during operation time. Wearing course provides pavement sealing and its surface should ensure safe traffic conditions. There are different technologies for constructing waterproofing layers and the most frequently used include [5]:

- asphalt based waterproofing:
  - torch-on membranes;
  - self-adhesive membranes;
  - asphalt mixtures;
  - conventional mastic waterproofing (grain size <4 mm);
- synthetic waterproofing:
  - polyurethane spray waterproofing;
  - methacrylate-methyl waterproofing;
  - polymer cement mortar waterproofing;
  - softened epoxy resin waterproofing.

In Poland the technical requirements for asphalt mixtures and the conditions of their production have been developed by the General Directorate for National Roads and Motorways (GDDKiA) in the form of Technical Requirements (WT-2). These requirements are often verified in recent years and the most recent draft version of the 2013 [6] does not recommend the use of polymer modified bitumen in mastic asphalt (MA), that requires high production temperatures. Polymer modified bitumen is replaced by multigrade binder 35/50. The following materials are used to construct bridge wearing courses [6]:

- mastic asphalt MA 8 and 11 (with binder 35/50 or multigrade binder 35/50);
- stone mastic asphalt SMA 5, 8 and 11;
- asphalt concrete for very thin layers BBTM 8 and 11;
- asphalt concrete AC 16S.

In stone mastic asphalt (SMA), asphalt concrete for very thin layers (BBTM) and asphalt concrete (AC) mixtures it is required to use the following polymer modified bitumens: PmB 45/80-55, PmB 45/80-65, PmB 65/105-60. The natural asphalt additive "Trinidad" in an amount of 1.5 to 2.0% by weight of the bitumen is recommended to use for the wearing course, which improves resistance to permanent deformation, limits aging processes, increases resistance to fatigue and low temperature and increases roughness [4].

2 Material and technological solutions of bridge waterproofing and pavements

Warsaw University of Technology in the years 2011-2013 conducted research project “Material and technological solutions of bridge waterproofing and pavements”. The aim of the project was the analysis of technical solutions of waterproofing and pavements on bridge concrete and steel decks, with particular emphasis on the impact of high technological temperatures on the durability of the materials used for constructing waterproofing and pavement layers. Asphalt binders used for pavement and waterproofing should have such properties that they do not crack at low temperatures in winter and do not go to the plastic state during the summer. Viscoelastic properties of asphalt binders in a wide temperature range can be achieved by modifying the bitumen with polymers. The most commonly used type of elastomeric polymers are in the form of a styrene-butadiene-styrene SBS copolymers.

2.1 The impact of high temperature on the bridge waterproofing destruction

In Poland, in recent years, polymer-modified materials are used for waterproofing of bridge decks.

2.1.1 Torch-on membranes

Waterproofing made of torch-on membrane can be easily damaged during installation by coating mass overheating with the burner flame. It should be noted that the temperature at the outlet of the burner flame reaches approximately 1000°C, and the process requires heating in a suitable distance. Overheating of the coating mass breaks the polymer bonds, which existence guarantees the good properties of polymer modified bitumen [7].
Research conducted at Warsaw University of Technology on the destructive impacts of high temperatures on materials containing polymer modified bitumen led to the formulation of interesting statements about the behaviour of torch-on membranes at high temperatures. Thermal annealing at temperatures of 200, 250 and 300°C of torch-on membranes containing polymers in its composition showed partial or complete degradation of the coating mass and reinforcement. Figure 3 shows the view of membrane annealed at 300°C.

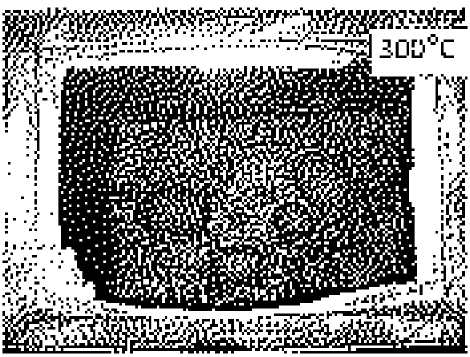

Figure 2 Bridge torch-on membrane annealed for 1 hour at 300°C

Membranes manufacturers quote the minimum temperature for SBS modified torch-on membranes of 100°C, at which no damage to the membrane should appear. In case of torch-on membranes containing polymer modified bitumen it should be noted that the maximum allowable temperature at which they can be used with no physical deterioration has not yet been determined. It is desirable that all SBS polymer modified torch-on membrane manufacturers defined for their products the maximum thermal resistance limits.

The results of torch-on membrane flexibility research conducted according to PN-EN 1109 at a temperature of -30°C showed that the membrane after annealing at 200°C has been damaged. The membrane upper surface showed cracks of the top layer of the coating mass up to the surface of the reinforcement. Higher temperatures (of 250 and 300°C), resulted in membrane cracks through the thickness of the coating weight and the reinforcement.

2.1.2 Mastic waterproofing

Asphalt mixtures for the bridge waterproofing during the production, transport and paving may be exposed to high temperatures that excess the temperatures required for technological processes. Moreover, the mastic waterproofing is again heated up when mastic protective course, produced at 200-240°C, is placed. The impact of high temperatures above 200°C can result in changes in the properties of binders and thus changes the viscoelastic properties of asphalt mixtures.
In order to determine the impact of high temperature on the functional properties of mastic bridge waterproofing, the tests of the traditional mastic of grain size up to 2 mm were carried out. Mastic tolerance to high temperatures during manufacturing, transport and paving was determined while warming at 200, 250 and 300°C. Thermal annealing time was 1 hour. For comparative purposes of the tested mastic samples the binder 35/50 and polymer modified bitumen PmB 45/80-55 were used.
In order to assess the consistency changes of asphalt mixtures (traditional mastic, mastic asphalt) under high temperature of 200, 250 and 300°C, penetration tests were carried out with indentation penetrometer in accordance with EN 12697-20.This test determines the depth of penetration in the traditional mastic or in mastic asphalt, under the force of 525 N, transmitted through a cylindrical indentor pin with a circular flat ended base of 500 mm^2. Load time is up to 60 minutes. Penetration (hardness) test results of mastic waterproofing as a function of temperature is shown in Figure 3 and 4.

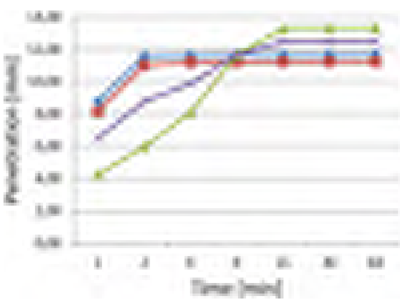

Figure 3 The impact of high temperatures on the changes of the penetration (hardness) of the mastic waterproofing with bitumen 35/50

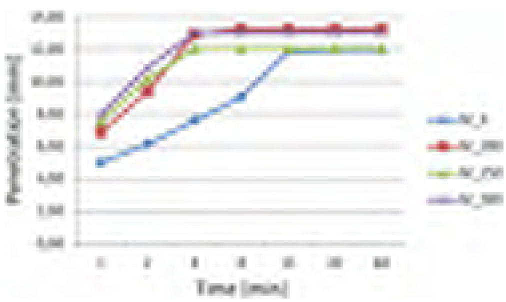

Figure 4 The impact of high temperatures on the changes of penetration (hardness) of the mastic waterproofing with polymer modified bitumen PmB 45/80-55

The results of the penetration of the annealed at high temperatures mastic asphalt with binder 35/50 (Figure 3) lead to the conclusion that both annealed at 200°C and not annealed mastic shows similar hardening rate. At temperatures of 250 and 300°C a large scale hardening can be observed, most likely due to changes in the bitumen group constitution (resins, oils, and asphaltenes). Penetration test results of mastic with polymer modified bitumen PmB 45/80-55 (Fig. 4) shown that mastic containing not annealed binder has initially low penetration of about 5 mm. Mastic with polymer modified bitumen annealed at 200, 250 and 300°C shows a significant increase in penetration due to a loss of elastic properties caused to loss of polymer bonds network, which was destroyed by high temperatures.

2.2 The impact of high temperature on the protective layer destruction

The high temperature of mastic asphalt (MA) used as a protective layer of the waterproofing membrane, causes dissolution of binder located in the waterproofing and penetration of this binder into the mastic asphalt and thus, a reduction in the thickness of the waterproofing layer. The impact of high temperature during laying the protective layer of mastic asphalt on the waterproofing membrane also causes melting of the polymer contained in the waterproofing, which penetrates into the mastic asphalt layer [7].
In order to determine the impact of a high temperature of 200, 250 and 300°C on the functional properties of the 8 mm grain size mastic asphalt protective course, penetration tests were carried out in accordance with EN 12697-20. Binder 20/30 or polymer modified bitumen PmB 25/55-60 were used in mastic asphalt. Annealing time of the mastic asphalt was one hour. Penetration (hardness) test results of the mastic asphalt MA8 containing mentioned binders are shown in Figures 5 and 6.

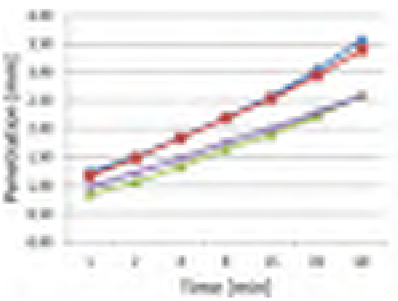

Figure 5 The impact of high temperatures on the changes of the penetration (hardness) of the mastic asphalt MA8 with bitumen 20/30

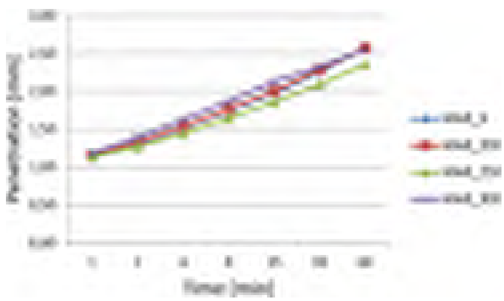

Figure 6 The impact of high temperatures on the changes of the penetration (hardness) of the mastic asphalt MA8 with polymer modified bitumen PmB 25/55-60

The penetration test results of mastic asphalt with bitumen 20/30 shown in Figure 5 indicate that the mastic asphalt (MA) is hardened slightly by changes in the bitumen group constitution and a partial transition of asphaltenes into resins. Hardening of the mastic asphalt both annealed at 200°C and not annealed is similar. Larger rate of hardening of mastic asphalt with binder 20/30 is observed after annealing at 250 and 300 °C.
The impact of high temperature on hardness change of mastic asphalt with polymer modified bitumen PmB 25/55-60 shown in Figure 6. Hardness test results show little variation of penetration, which means that the impact of high temperatures affects the hardness of mastic asphalt than traditional mastic to a lesser extent. Mastic asphalt contains less binder and more grits than the traditional mastic, so the effect of the deterioration of the elastic properties of polymer modified bitumen is not apparent.

3 Conclusions

The widespread use of asphalt pavement and waterproofing materials containing polymers creates the danger of damage to bridge pavements due to the impact of high technological temperatures. High temperatures above 200°C result in the destruction of polymer bond network in materials containing polymers, which is the cause for the loss of the durability of waterproofing or the pavement. As the right direction of the works over the quality of pavement systems a review of requirements concerning the bridge pavements should be considered to eliminate the use of polymer modified bitumen. Eliminating the use of polymer modified bitumen in mastic asphalt for protective course and the recommended use of bitumen 35/50 and multigrade binder 35/50, allows using high-temperature annealing up to 240°C. Such mastic asphalts used for protective layers will be highly durable and have good technical properties.
Even the use of the highest quality materials, with poor workmanship does not guarantee durability of the waterproofing systems. This statement can be supported by the opinion quoted below [8]: "Bridges built in Denmark are characterized by their pavement durability. One reason for this is to comply with stringent technical requirements, extensive control of materials as well as thorough supervision of the works."

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References

[1] Hulsey, J. L., Yang, L., and Raad, L.: Wearing Surfaces for Orthotropic Steel Bridge Decks, Tansportation Research Record 1654, Paper No. 99-1049, Transportation Research Board, Washington DC, 1999.

[2] Bocci, E. and Canestrari, F.: Analysis of Structural Compatibilityat Interface Between Asphalt Concrete Pavements and Orthotropic Steel Deck Surfaces. Tansportation Research Record 2293: Journal of the Transportation Research Board, Asphalt Materials and Mixtures, Vol. 1., 2012.

[3] Sprinkel, M. M. and Apeagyei, A. K.: Evaluation of the Installation and Initial Condition of Overlays on Bridge Decks Rosphalt. Virginia Center for Transportation Innovation and Research. Final Report VCTIR 13-R5. June 2013.

[4] Piłat, J., Radziszewski, P.: Asphalt Pavements, WKŁ, Warsaw 2010.

[5] Catalog of the surface protectives for the road objects. Road and Bridge Research Institute (IBDiM), Warsaw 2002.

[6] Asphalt pavements for national roads. Asphalt mixtures. Technical Requirements WT-2 2013 DRAFT. General Directorate for National Roads and Motorways (GDDKiA), 2013.

[7] Danowski, M.: Experience with insulation and pavements on bridge structures – elaboration on the basis of article Hermann W. Fritc i Sivotha Hean: „Erfahrungen mit Brückenabdichtungen und Brückenbelägen." Nowości Zagranicznej Techniki Drogowej No 141, pp. 85-104, 2000.

[8] Kwapiński, M.: What causes the bridge pavements in Denmark are such durable? – elaboration on the basis of article Vibeke Wegan: „Bridge pavements – what makes them long-lasting?" Nowości Zagranicznej Techniki Drogowej, No 142, pp. 89-94, 2000.

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EFFECTS OF CLIMATIC FACTORS ON THE SHAPE OF DEFLECTION BOWL

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Abstract

Although the pavement structures can be characterized by several condition parameters and thus qualified, some kind of load-bearing capacity parameter is one of the most important features. Nowadays there are many theoretical approaches and practical methods to define but the detection of the deflection of the structure caused by vertical load is still the most widely used. Presently the dynamic falling weight equipment is not only becoming more common but it is accepted in addition to the previous and widely recognized, so-called Benkelman beam method. This type of equipment called FWD is available in Hungary since the 90's. It is well known that this device can record not only the deflection under load, but the deflections also at arbitrary distance from load axle. So the new overlay design method that is currently under development by Hungarian Road Administration has been established based on this tool and, contrary to the previously used on, it experienced, requires the recording of the deflection bowl.
However, the deflection bowl and the shape factors describing it depend not only on the load-bearing capacity of the pavement structure and the magnitude of load force, but also the measurement conditions also, especially the climatic conditions during the measurement (e.g. temperature, precipitation conditions) have significant impacts on it. Because of these effects are quite environment specific, it is advisable to study their impacts through a more thorough examination of trial section so the distorting effect of these factors can be eliminated. In this paper, evaluations of four-year of meteorological data and deflection measurement series on a Hungarian road section were carried out to examine the change of the deflection bowl's parameters at different time points during the tests. In particular, the study focuses on the development of a correction factor that would be made based on the evaluation of the measuring results.

Keywords: pavement deflection, subgrade modulus, temperature, precipitation, seasonal effect

1 Introduction

The application of seasonal coefficient during the evaluation of deflection data is very important. The bearing capacity is one of the most important condition parameters of the pavement structures. This definition refers to a theoretical limit, showing the unsuitable condition for its intended use. It follows that this value cannot be measured directly in practice. Although today several measurement methods are available to make conclusions on the actual load-bearing capacity of the pavement structure, the deflection measurement is the most common method. The pavement condition can be characterized just limitedly by a single deflection value it, may even be misleading, so it would be more appropriate to record the whole deflection bowl. This problem has been overcome by the development of measurement method and appea-

rance of falling weight deflectometer and that is widespread in the recording of the deflection curve. Two external factors complicate largely the study of the relationship between the deflection bowl and the load-bearing capacity of pavement.
One of the parameters is the strong temperature dependence of pavement layer. The instantaneous temperature of asphalt layers affects significantly the shape of deflection bowl. Several methods are available to overcome this effect, but the complexity of the problem does not allow the creation of a universal formula. The other important effect is the condition of the subgrade. The deflection measurement examines the integrated condition of pavement and subgrade, thus the condition change of subgrade influences the measured deflection data. Fig. 1 illustrates the influence of these two dominant effects on the deflection bowl.

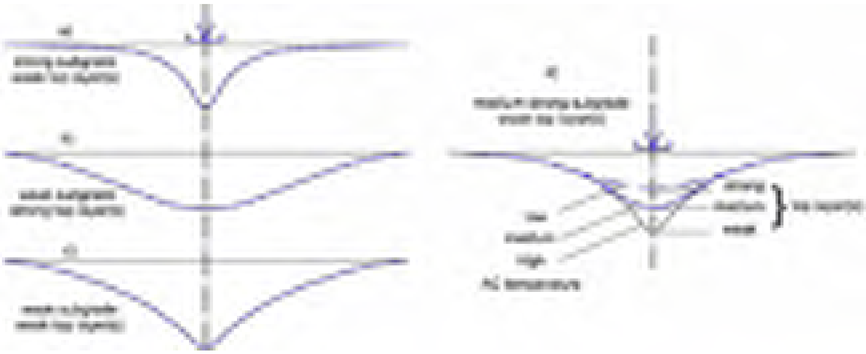

Figure 1 Deflection bowls of different pavement structure types [1]

Although this effect depends on the geographical location, but the period, when the highest deflection values can be measured because of the weak bearing capacity of subgrade after the wetting, occurs at the late winter snowmelt and the spring rains in the typically European climatic condition. This period is considered critical in terms of the remaining service life of pavement and these deflection values are considered as characteristic for the performance of structure. This critical period is also called as “spring deflection” in the literature. The deflection measurements are concentrated in this period in 60’s and 70’s, but later this measurement period extended to the summer and autumn months. As a result, the use of seasonal has seemed to be necessary.
According to the current Hungarian Standard [2], the deflection measurements must be done in the most unfavourable spring period March to May by reason of wetting before the year of overlaying. The results measured other periods must be converted for the most adverse period. The determination of correction factor is recorded in a Hungarian Standard [3]. It can be calculated using monthly measured deflection values at least for a year on a similar pavement structure, and comparable soil and hydrological condition, between 1 March and 15 June measured values divided by the values recorded at the time of measurement.
If it is not possible, correction factor’s approximate values can be taken of Table 1.The seasonal factors can be obtained depending on the type of soil and the date of measurement. The value of seasonal multiplier equals to 1.0 in the characteristic period. The coefficients are good for the case of dry and wet region and hot mix asphalt layers above 100 mm total thickness, in other cases the values in brackets on Table 1 can be used.
However, the observed changes of climatic condition in the past decade have raised the need to update this element of Hungarian Regulation. Accordingly, the standards draft under development has a new approach. The correction of the standard subgrade condition is recommended to perform on the basis of the precipitation before measurement with the correction values indicated on Table 2. It means that the deflection values should be increased with the values on Table 2 that depend on the precipitation amount of previous two months (60 days) and the soil type.

Table 1 The seasonal correction factors [2]

Soil classification	The month of deflection measurement				
	April	May	June, July	August, September	October, November
I-II.	1.0	1.0	1.0	1.0	1.0
III.			1.1	1.1	1.2
IV-V.		1.1	1.3 (1.4)	1.5 (1.6)	1.5 (1.6)
VI-IX.	1.1	1.0	1.1 (1.2)	1.2 (1.4)	1.3 (1.5)

Table 2 Proposed correction values of the critic subgrade condition [4]

Soil class.	Negative difference of 60-day precipitation balance before deflection measurement multi-year average				
	No or positive difference	Below 10%	Below 20%	Below 30%	More than 30%
I-II.	1.0	1.0	1.0	1.0	1.1
III.	1.0	1.0	1.1	1.1	1.2
IV-V.	1.0	1.2	1.4	1.5	1.6
VI-IX.	1.0	1.1	1.3	1.4	1.5

This paper examines how close relationship can be observed between the precipitation conditions before measurement and the subgrade modulus estimated from FWD results. First, the deflection measurement results carried out every hour at the same cross-section in spring and summer reviewed, in order to estimate the effect of daily temperature fluctuation on subgrade modulus. Then it will be examined which relationship can be found between the measured data and the meteorological conditions of the period before measurement by using deflection series measured on Hungarian motorway traffic lanes.

2 The relationship between the temperature and the shape of deflection bowl

It is known that the temperature affects pavement layers moduli, thus also its load-bearing capacity and deflection. Earlier the relationship between the air and pavement temperature and deflection was investigated. The measurement was carried out by FWD device, therefore, it could be analysed the relationship between the temperature and not only the central deflection, but the whole deflection bowl. The measurements were carried out between 6 a.m. and 7 p.m. in April and August.

Fig. 2 shows the hourly deflection results. It can be seen that the deflection measured at any distance from load axle continuously progressively increased as temperature rise during the day, and then slowly reduce. The rate and extent of increasing and decreasing are not the same, because the changes of air and pavement temperature are at the same either. Difference of approximately 40% between the maximum and the minimum value of the central deflection has been experienced on the test days. The difference between the maximum and minimum values decreases gradually moving away from load centre. This difference can be even 15-18 % at the distance of 600 mm, while the fluctuation within a day is less than 10% at 900 mm and greater distance.

Temperature correction is generally used just for the deflection of first 4-5 sensors. Jansen developed correction formula for the deflection in 0, 200, 300, 450, 600 mm from load centre [5]. If the deflections (Fig. 2) under back sensors measured in April and August are compared, it can be seen that there is no relevant difference between the values, which is confirmed, that the temperature correction is not necessary for the deflection above 900 mm distance.

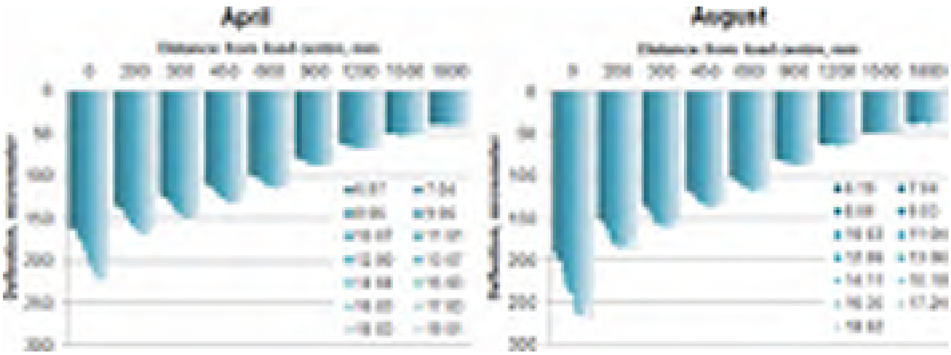

Figure 2 Hourly measured deflections

If the deflection bowls are plotted as a function of hourly recorded deflection, it can be seen that the shapes of the bowls are almost the same from the distance 900 mm. Many conclusions can be drawn for each pavement layer using the parameters (SCI, BDI, BCI) deduced from the shape of deflection bowl. While the difference between D0 and (typically) D300 provides information about the condition of top layer, the bearing capacity of subgrade can be estimated from the deflection values measured in a distance 900 mm or more. Hereafter the paper deals the relationship between the subgrade condition and deflection bowl.

3 The evaluation of subgrade condition using FWD measurement results

The performance of flexible pavements under load can be examined by the Boussinesq-formula with a good accuracy approximation, [6]. However, it is the generally accepted relationship that the modulus at distance “r” from the load centre is the same as the modulus of layer in depth “z=r” under the load centre.

$$E_{eq(r)} = \frac{\left(1-\nu^2\right)\sigma_0 a^2}{r * d_0} \tag{1}$$

where:

$E_{eq(r)}$ equivalent surface modulus at distance r from the load centre, MPa;
r distance from load centre, mm.

German researchers [7] analysed the currently used evaluation correlations of FWD test. The surface modulus calculated from the deflection at the distance of 1200 mm from load axle has chosen as the best characteristic of subgrade bearing capacity. It can be determined on the basis of Eq. (1). Jendia proposed the application of the deflection values of sensors in higher distance from load centre [8]. Jendia introduced the subgrade indicator (UI) as a definition.
According to Swedish research works, the subgrade modulus can be properly estimated by the following formula [9]:

$$E_{subgrade} = \frac{52000}{d_{900}^{1.5}} \tag{2}$$

4 The subgrade modulus test series on a motorway section

The proposed Hungarian regulation takes into account the seasonal fluctuations because of the climatic effects with the correction in Table 1. However, just few measurement results are available to validate of these values; therefore the results of measurements carried out for other purposes were also examined here.

For four years, every six months the deflections of a Hungarian motorway section were compared with the precipitation of the given period. The FWD measurements were carried out in each (travel, overtaking lane, paved shoulder) lane, every 100 m, 45 cross-sections per lane (4500 m). Two-two measurements were done in November and December, and others in summer months. 6 million ESAL (100 kN) ran on the travel lane during the investigation. According to the weather station close to the section, the total amount of rain was 3000 mm. The temporal distribution of the last five years precipitation in each month is shown on Table 3.

Table 3 Cumulative monthly precipitation amount, [mm]

Year/Month	2009	2010	2011	2012	2013	Average	Total
1	53.6	82.9	22.9	30.9	58.8	49.82	249.10
2	48.2	71.5	15.0	22.9	84.5	48.42	242.10
3	48.8	20.9	43.5	0.4	92.8	41.28	206.40
4	1.8	71.7	20.3	18.5	19.9	26.44	132.20
5	41.1	160.6	40.9	23.6	106.8	74.60	373.00
6	84.5	83.9	59.0	95.0	61.5	76.78	383.90
7	34.9	87.8	57.6	98.3	0.1	55.74	278.70
8	46.8	77.9	5.3	1.1	39.0	34.02	170.10
9	14.4	95.5	1.1	48.5	24.7	36.84	184.20
10	28.3	24.7	14.9	77.0	42.2	37.42	187.10
11	89.2	73.3	0.0	15.4	42.7	44.12	220.60
12	43.6	121.7	69.9	48.9	1.1	57.04	285.20
Total	535.2	972.4	350.4	480.5	574.1	582.52	–

Extremely rainy months were not observed. The maximum of cumulative precipitation amount of the test period is in June, although this month was not the wettest in any of the five years. This is slightly different from our expectations, because April or May is considered the wettest months. Another interesting anomaly is that the wettest month of 2012 was the driest in 2013. These results did not confirm to the existence of the critical spring period, therefore the revision of Table 1 is justifiable. It is noted, that the impact of late winter snowmelt has not been taken into account. However, the deflection measurements were performed between May and December, so this effect does not influence the following results.

Many options are available to estimate the load-bearing capacity of pavement layers, as shown in Section 2. Other correlations also use D_{900} or D_{1200}. These values have strong correlation with each other, so the determined correspondences would be justified for the application of other estimation formulas of subgrade modulus. The estimated subgrade moduli were determined by Eq. (2), after that the characteristic was calculated. The strength of the relationship between subgrade moduli and the rain amount of 30, 60, 90 days before the measurement was examined. In the case of the overtaking lane, the subgrade moduli had a close correlation with the precipitation amount of 30 or 60 days, but this close relationship was not detected for 90-day values, Fig. 3 and Table 4 s that. The correlation between the 30-day precipitation and the subgrade moduli of paved shoulder is detectable, but it was not clear neither in the case of 60 and 90-day data, nor the results of travel lane.

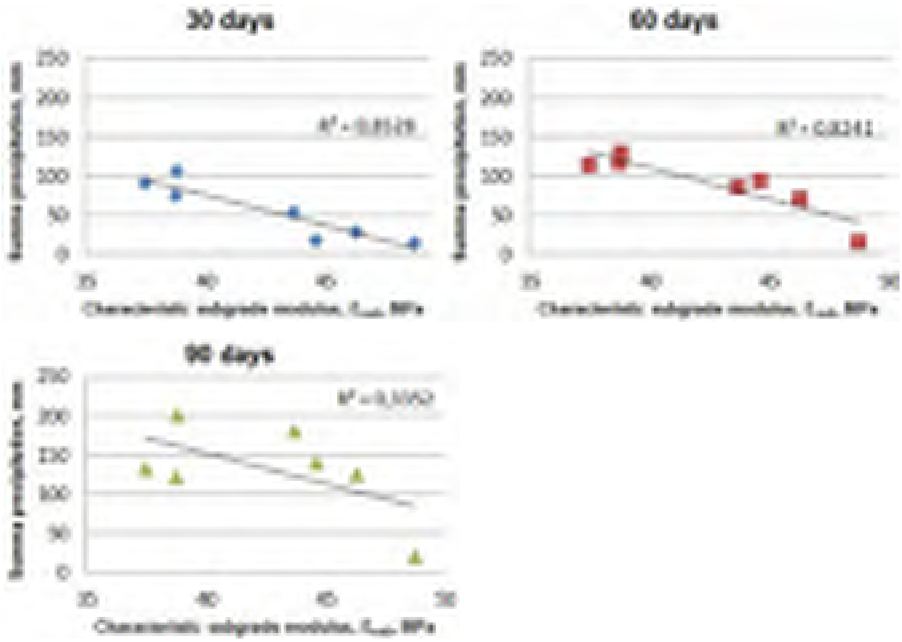

Figure 3 Correlation of overtaking lane

Table 4 Correlation between cumulative monthly precipitation amount and subgrade modulus

Number of examined days	Overtaking lane	Travel lane	Paved shoulder
30	0.85	0.21	0.64
60	0.82	0.12	0.38
90	0.34	0.00	0.08

In accordance with the above mentioned close relationship between the subgrade moduli based on deflection bowl and cumulative precipitation amount of the days prior to the measurement can be detected only under certain conditions. The results show that the closest connection was in the case of 30-day period before measurement, and worst values arose in the 90-days period. Furthermore, it is an important statement that the highest correlation was seen at the overtaking lane and the paved shoulder. Both lanes are connected directly to the unpaved shoulder and central reserve with one of their edges helping the direct entering of rain under these traffic lanes obviously. In case of paved shoulder, this connection is influenced by the condition, size and slope of shoulder, the height of embankment, etc. In our opinion, these factors affect the degree of wetting of paved shoulder together, so it results in weaker relationship. In this regard the situation of travel lane is special because it is not connected directly to the central reserve thereby it reacts slightly the precipitation preceding the measurement that the estimated moduli are not correlated with the cumulative 30-day amount of rain. It increases the importance of this statement that the traffic load of this lane is the characteristic in the course of overlay design. The temperature correction developed on this traffic lane does not correct but even worsens the results of load-bearing capacity.

5 Conclusion

The current Hungarian overlay design method is under revision, and our investigation shows that it is possible to convert the deflection results to "spring deflection" using factors depending on the actual month. The meteorological data and the accumulated experience show that the distorting effect of this approach can be significant.
In accordance with a new approach, the seasonal fluctuations of load-bearing capacity of pavement would be advisable to correct it using the amount of precipitation before measurement, it influences the subgrade modulus. The associated tests were done on low traffic load, 2*1 lane road with low height of embankment. The aim of this paper is the investigation that the proposed corrections can be applied if the drainage of motorway section functions well and the section is located on embankment.
In accordance with our results, it is verifiable that there can be a close relationship between the characteristic subgrade moduli and the previously cumulative precipitation. Although, based on our analysis, the relationship is closer in the case of the amount of 30-day precipitation than the 60-day one. The implied water movement will be different on a multi-lane motorway located on high embankment from that based on the behaviour of minor roads. Our proposal is that the correction mentioned could not be used for design of travel lane of motorway; separate correction should be created for it.

References

[1] Jung, F. W.: Numerical Deflection Basin Interpretation and Temperature Adjustment in Non-Destructive Testing of Flexible Pavements, Ministry of Transportation, Ontario, 1988.

[2] ÚT 2-1.202. Design of Road Pavement Structures and Overlay Design with Asphalt Surfacings, (in Hungarian) Hungarian Road Society, 2005.

[3] MSZ 2509/4-1989 Bearing capacity test on pavement structures. Deflection measurement, (in Hungarian) Hungarian Standard Committee, 1989.

[4] Karoliny, M.: Load-bearing capacity of road, the influences of subgrade (in Hungarian) http://utugyilapok.hu/cikkek/kozlekedesi-palyak-teherbirokepessege-a-foldmu-hatasai/

[5] Jansen, D.: Temperaturkorrektur von mit dem Falling-Weight-Deflectometer gemessenen Deflexionen auf Asphaltbefestigungen, PhD Dissertation, Duisburg, 2009.

[6] Ullidtz, P.: Modelling Flexible Pavement Response and Performance, Polyteknisk Forlag, Lyngby, 1998.

[7] Hothan, J. & Schäfer, F.: Analyse und Weiterentwicklung der Bewertung von Tragfähigkeitsmessungen, Strasse und Autobahn, 7, 2004.

[8] Jendia, S.: Bewertung der Tragfähigkeit von bituminösen Straßenbefestigungen, PhD Dissertation, Karlsruhe, 1995.

[9] Christiansen, A.S., Holst, M.L. & Baltzer, S.: Pavement Performance Models, Nordic Cooperation Program, Report No.2, 2010.

SUBGRADE BEARING CAPACITY INFLUENCE ON FLEXIBLE PAVEMENT STRUCTURES BEHAVIOUR

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Abstract

The design methodology of a flexible pavement structures is extremely laborious and complex requiring several work steps. Thus initially are required conducting field studies on the composition and volume of traffic, geotechnical characteristics of subgrade soil, hydrological regime of pavement system. Then follow the itself design of the pavement structure which involves the following steps: establishing the calculation traffic, determining the bearing capacity of the road bed, the choice of an embodiment of the pavement structure, pavement structure analysis at the standard axle loading, establishing the pavement structure behaviour under the traffic. In general, in the pavement design practice is considered that the choice of a pavement structure embodiment with thicknesses as great for pavement layers and constituent materials as best ensures a good behaviour of in service pavement structure. Unfortunately is overlook the variation of the subgrade soil bearing capacity during a year due to variations in environmental conditions (humidity, temperature, freeze-thaw) specific for the road site. This paper aims to highlight the magnitude of the subgrade bearing capacity influence on the flexible pavement structures behaviour. In the study it will be shown that the stress and strain state in the pavement structure is a very sensitive indicator of the importance of road bed soil condition and quality, and as such, it is very important to take this aspect into account in order to have not surprises during the pavement exploatation.

Keywords: flexible pavement structures, subgrade soil, bearing capacity

1 Introduction

The design methodology of a flexible pavement structures is extremely laborious and complex. In general, in the pavement design practice is considered that the choice of a pavement structure embodiment with thicknesses as great for pavement layers and constituent materials as best ensures a good behaviour of in service pavement structure. Unfortunately is overlook the variation of the subgrade soil bearing capacity during a year due to variations in environmental conditions (humidity, temperature, freeze-thaw) specific for the road site.
At international level there are numerous studies regarding the subgrade bearing capacity influence on flexible pavement structures behaviour [1, 2, 3]. For example, to characterize the subgrade soil behaviour during a year, in states such as Washington and Minnesota from USA are used seasonal adjustment coefficients for subgrade elasticity modulus value (Pierce and Mahoney, 1996 [1]).
Aim of this paper is that based on numerical simulations performed on a computer program to analyze how the loss of subgrade bearing capacity affects the operational behaviour of pavement structures. Will be determined the strain level in the critical points of two flexible pavement structures for five possible operation situations. The CALDEROM 2000 computing program used, the composition of flexible pavement structures analyzed, the calculation

vehicle and traffic classes are the ones specific for Romanian norms in force PD 177 [4] and CD 155 [5].
The input data used in the CALDEROM 2000 calculation program are the thickness of the layers, the material deformation characteristics of road layers (the dynamic elasticity modulus and Poisson's ratio) and the 115 kN standard axle characteristics (the contact pressure and the radius of the contact surface between the road and tires). The results are the full stress and strain state conforming to the Burmister model [6] for the analysis of pavement structures.

2 Calculation assumptions

This paper aims to highlight the magnitude of the subgrade bearing capacity influence on the flexible pavement structures behaviour. For this purpose have been taken into account two pavement structures: one with a structure allowing the movement of a Light traffic class and one for a Heavy traffic class. The composition of the analyzed flexible pavement structures, as well as the characteristics of their component materials are presented in Tables 1 and 2.

Table 1 Characteristics of the pavement structure for Light traffic (PS1).

Material in pavement structure layer	Layer thickness, h [cm]	Dynamic elasticity modulus, E [MPa]	Poisson's ratio, μ
Asphalt concrete, BA 16	4	3600	0.35
Bituminous coated, AB 2	8	5000	0.35
Granular material, Ballast	15	191	0.27
Subgrade soil	∞	100	0.27

Table 2 Characteristics of the pavement structure for Heavy traffic (PS2).

Material in pavement structure layer	Layer thickness, h [cm]	Dynamic elasticity modulus, E [MPa]	Poisson's ratio, μ
Asphalt concrete, BAR 16	4	3600	0.35
Binder, BAD 25	5	3000	0.35
Bituminous coated, AB 2	8	5000	0.35
Granular material, Crushed stone	12	400	0.27
Granular material, Ballast	15	191	0.27
Subgrade soil	∞	100	0.27

In Tables 1 and 2, the deformation characteristics of the pavement structure component materials and the deformation characteristics of the subgrade soil have design values according to the Romanian norm PD 177-2001 [4]. In Table 3 are presented the traffic classes from Romania based on standard axle load of 115 kN used in pavement structures design.

Table 3 Traffic classes according to Romanian norm CD 155-2001 [5].

Traffic class	Traffic volume, Nc [millions of standard axles of 115 kN]
Exceptional	3.0…10.0
Very Heavy	1.0…3.0
Heavy	0.3…1.0
Middle	0.1…0.3
Light	0.03…0.1
Very Light	‹ 0.03

The characteristics of the standard axle load of 115 kN are as follows:
· twin wheels load: 57.5 kN;
· contact pressure: 0.625 MPa;
· the radius of the contact surface: 17.1 cm.

In operation, due to the traffic loads, the environmental conditions variation (humidity, temperature, freeze-thaw), the possible problems of execution and a poor quality of road maintenance, the subgrade bearing capacity can decrease. Table 4 shows how the decrease of the subgrade dynamic elasticity modulus value immediately influences the value of ballast module from the subbase course, according to the Romanian design norm PD 177-2001 [4].

Table 4 Assumptions for calculation.

Computing hypothesis	I	II	III	IV	V
Ep [MPa]	100	80	60	40	20
Eb [MPa]	191	153	114	76	38

Where:
Ep the subgrade soil dynamic elasticity modulus;
Eb the dynamic elasticity modulus of ballast.

Also in Table 4 are presented the five calculation assumptions corresponding to the subgrade dynamic elasticity modulus values, on which will be developed the present study. In order to establish the influence of the subgrade bearing capacity on flexible pavement structures behaviour, will be analyzed the two pavement structures (PS1 and PS2) in the five possible situations adopted. To determine the state of stress and strain in the five calculation situations was used the computer program CALDEROM 2000, that is integral part of the “Normative for flexible and semi-rigid pavement structures design” [4]. CALDEROM 2000 program is based on analytical solving of the stress and strain state of a pavement structure under load, by using the Burmister multilayer elastic model [6]. In the Burmister model, pavement structure is modeled by linear elastic isotropic and homogeneous layers infinite in plane, of finite thickness, with the exception of semi-infinite subgrade soil, and tire loading are treated as circular static loads, exerted as vertical efforts (the vehicle weight).

3 The study results

In the present study it was intended the estimation of flexible pavement structures response in their critical points:
· ε_r = the horizontal tensile strain at the base of bituminous layers;
· ε_z = the vertical compressive strain at the level of road bed.

The analysis results of flexible pavement structures PS1 and PS2 in the five hypotheses are presented in Tables 5 and 6. The analysis results of the strain state variation in the flexible pavement structures PS1 and PS2, conducted with the program CALDEROM 2000, are presented graphically in Fig. 1 and 2.Analyzing the strains variation in the critical points of the pavement structures, are found the following increases towards computing hypothesis I (the ideal situation) taken as a reference (Tables 7 and 8). From Figs. 1 and 2, and the analysis of tables 8 and 9 results that the subgrade bearing capacity variation influence is higher on pavement structures with smaller thicknesses. For example, for a subgrade bearing capacity decrease of 80%, is found that the magnitude of the horizontal tensile strain increase at the base of bituminous layers is about double size (79 %) for PS1 pavement structure (of smaller thickness) towards that observed for PS2 pavement structure (of higher thickness) (41 %).

Table 5 CALDEROM 2000 results for pavement structure PS1.

Computing hypothesis	Horizontal tensile strain at the base of bituminous layers, ε_r* [microdef.]	Vertical compressive strain at the level of road bed, ε_z* [microdef.]
I	272.00	967.00
II	300.00	1110.00
III	338.00	1310.00
IV	393.00	1630.00
V	488.00	2330.00
* absolute values		

Table 6 CALDEROM 2000 results for pavement structure PS2.

Computing hypothesis	Horizontal tensile strain at the base of bituminous layers, ε_r* [microdef.]	Vertical compressive strain at the level of road bed, ε_z* [microdef.]
I	170.00	456.00
II	179.00	520.00
III	191.00	611.00
IV	208.00	757.00
V	240.00	1050.00
* absolute values		

Table 7 Strains variation for the pavement structure PS1.

Computing hypothesis	Horizontal tensile strain variation at the base of bituminous layers, $\Delta\varepsilon_r$ [%]	Vertical compressive strain variation at the level of road bed, $\Delta\varepsilon_z$ [%]
I	0	0
II	10	15
III	24	35
IV	44	69
V	79	141

Table 8 Strains variation for the pavement structure PS2.

Computing hypothesis	Horizontal tensile strain variation at the base of bituminous layers, $\Delta\varepsilon_r$ [%]	Vertical compressive strain variation at the level of road bed, $\Delta\varepsilon_z$ [%]
I	0	0
II	5	14
III	12	34
IV	22	66
V	41	130

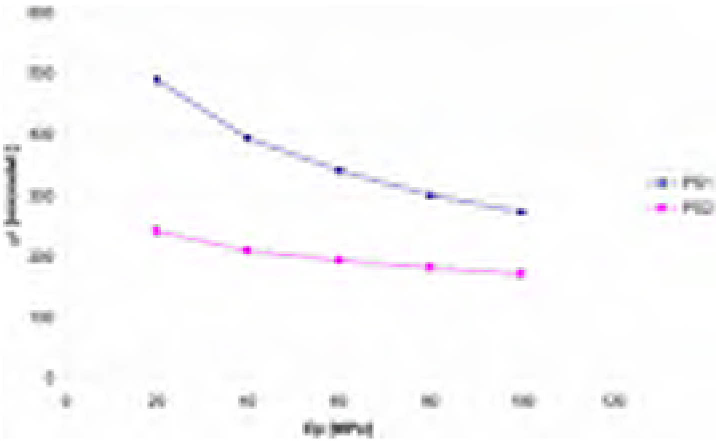

Figure 1 Horizontal tensile strain variation at the base of bituminous layers.

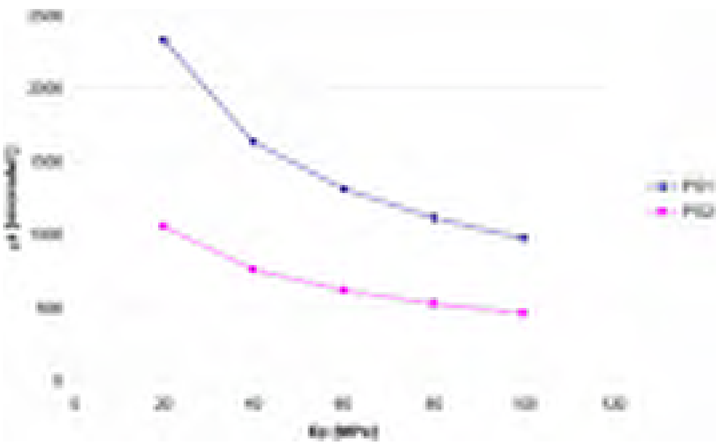

Figure 2 Vertical compressive strain variation at the level of road bed.

When is analyzed the subgrade bearing capacity influence on the compressive strain variation at the level of road bed, differences between the two pavement structures decrease. For example, for a subgrade bearing capacity decrease of 80 %, is found that the magnitude of the vertical compressive strain increase at the level of road bed differs with only 11 %, being registered (141 %) for PS1 pavement structure (of smaller thickness) towards that observed for PS2 pavement structure (of higher thickness) (130 %).
Comparing the tensile strain variation at the base of bituminous layers with the compressive strain variation at the level of road bed, is found that the subgrade bearing capacity influence is higher on compressive strain variation at the level of road bed.
It must shown that if in the hypothesis I (with better subgrade bearing capacity) the pavement structure PS1 could take over a Light traffic class and the pavement structure PS2 take over a class of Heavy traffic, then in the hypothesis V (corresponding to a decrease in subgrade bearing capacity by 80 %) was found the decrease of traffic class to Very Light for PS1, and respectively to Light for PS2. Thus, loss of subgrade bearing capacity entail the decrease of traffic values on which the pavement structures were designed to serve. In such conditions, if not repaired the problem then the pavement structures will be rapidly degrade.

4 Conclusions

This paper highlights the magnitude of the subgrade bearing capacity influence on the flexible pavement structures behaviour. Thus, after performing numerical simulations can be seen that the subgrade bearing capacity influence is higher on pavement structures with smaller thicknesses. Also, having regard the significantly size of the strain level growth in the critical points of pavement structures at the time of subgrade bearing capacity loss, can be emphasize the importance of execution works and roads maintenance quality for their behaviour in service.

In conclusion, the stress and strain state in the pavement structure is a very sensitive indicator of the importance of road bed soil condition and quality, and as such, it is very important to take this aspect into account in order to have not surprises during the pavement exploatation.

References

[1] Pierce, L.M. & Mahoney, J.P.: Asphalt Concrete Overlay Design Case Studies, Transportation Research Record 1543, Transportation Research Board, Washington, DC, 1996.

[2] Ovik, J., Birgisson, B. & Newcomb, D.E.: Characterizing Seasonal Variations in Flexible Pavement Material Properties, Transportation Research Record 1684, Transportation Research Board, Washington, DC, 1999.

[3] Thomas, J., Newcomb, D.E. & Siekmeier, J.: Foundation Requirements for Perpetual Pavements, Proceedings. Intl. Symp. on Design and Construction of Long Lasting Asphalt Pavements, National Center for Asphalt Technology, Auburn University, Alabama, pp. 263-283, 2004.

[4] Normative for flexible and semi-rigid pavement structures design. (Analytical method), indicative PD 177 2001.

[5] Departmental technical instructions on determining the technical state of modern roads, indicative CD 155 2001.

[6] Burmister, D.M.: The theory of the stress and displacements in layered systems and applications of design of airport runway, Proceeding of the Highway Research Board, 23, pp. 126 148, 1943.

LABORATORY AND FIELD EXPERIENCE WITH PMMA/ATH COMPOSITE DUST IN ASPHALT MIXTURES

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Abstract

Heavy loaded asphalt pavements must withstand formation of permanent deformations and cracking caused by temperature changes and fatigue. Ordinary paving grade bitumen has limited range of service temperatures. To expand range of service temperatures on most cases some costly polymeric materials like SBS are added. With this study we proved that addition of waste materials can be almost equivalent solution to improved asphalt properties. The PMMA/ATH – poly-methyl methacrylate (PMMA) filled with a fine dispersion aluminium trihydrate (ATH) is a composite used as a substitute for many indoor household ceramics. Processing and various polishing stages generate a considerable amount of waste powder. Mechanical properties of PMMA/ATH powder proved suitable for bitumen modification. Results of the study show that PMMA/ATH composite dust improves the performance of asphalt at high temperatures. Despite the advantages of PMMA/ATH composite dust on asphalt performance at high temperatures, it does not show a considerable influence on the low temperature performance of asphalt.

Keywords: asphalt mixture, composite dust, deformation, cracking, high temperature

1 Introduction

With reuse of waste materials in asphalt mixes environmental and economical advantage can be achieved by limiting the extraction of natural sources of material. But in such cases investors are mostly suspicious and they demand that performance of asphalt mix must be almost as good for the mixes made from traditional materials [1]. We managed to change a by-product, which was in former times considered as waste, to an additive for improving the performance of asphalt mixtures. PMMA/ATH composite dust is the waste material obtained by polishing of PMMA/ATH composite sheets. Final goal of our study was new asphalt mixture containing the PMMA/ATH composite dust. There are two ways to introduce PMMA/ATH dust as additive in asphalt mixture. First it was treated as additive to filler in asphalt mixtures (dry process) and second PMMA/ATH dust was treated as additive to bitumen (wet process). After first tests it was clear that the addition of dust even improves the quality of the asphalt layers. For dry process we had to determine the optimal ratio between ordinary filler and PMMA/ATH dust to achieved good mechanical properties of asphalt layer. Also for wet process optimal ratio between bitumen and PMMA/ATH dust had to be determined. We tried find out, if the bitumen modified with PMMA/ATH dust has similar properties as bitumen modified with other commercial additives. Several different asphalt mixtures containing PMMA/ATH dust were prepared in laboratory and several standard test methods were performed to evaluate asphalt mixtures (EN 12697-1, 2, 5, 6, 22, 30, 34, and 46). To address the human health and environment we assessed the risk for chemicals. We performed quantitative and qualitative analysis of dust and vapor, which are exhausted by heating of PMMA/ATH dust up to 180°C.

For 3 different types of asphalt mixtures industrial production was carried out, which means that we produced around 740 tones of asphalt mixtures, and laid them in a field test (normal road).

2 Input material and experimental work in laboratory

Industrial production of polymethylmethacylate/Aluminium hydroxide (PMMA/ATH) composite plates is taking place in Slovenia. These plates are due to high hardness, resistance to most chemical substances, mechanical and volume stability at low and high temperatures widely used for household ceramics and also outdoor. More than 50 wt. % of the plates consists of Aluminium hydroxide (Al (OH)3) which is chemically similar to Hydrated lime (Ca (OH)2). Hydrated lime is known as additive to improve adhesion between the binder and aggregate in asphalt mixtures [2, 3]. Similar effect was expected for (PMMA/ATH) composite materials. For asphalt mixtures prepared in laboratory paving grade bitumen B 70/100 and dolomite stone aggregate were used.

2.1 PMMA/ATH composite dust as additive to filler

First standard tests were performed to enable addition of PMMA/ATH composite dust as filler in asphalt mixture [4]. It was proved that adhesion between PMMA/ATH composite and bitumen is as good as expected. We had to assure that sieving curve (Tab. 1) and void content with Rigden test (Tab. 2) were in accordance with standardized requirements. We determined that weight ratio 5:1 between ordinary filler and PMMA/ATH composite dust is still in accordance with standardized requirements for filler.

Table 1 Sieving analyses according to method EN 933-10:2002 of ordinary filler, PMMA/ATH composite dust and their mixtures

Sieve [mm]	PMMA/ATH composite dust	Ordinary filler	Ordinary filler: PMMA/ATH composite dust = 5:1 wt.	Ordinary filler: PMMA/ATH composite dust = 8:1 wt.	Requirement according to EN 13043
2.00	100	100	100	100	100
0.125	74	98	94	95	85 – 100
0.063	41	89	81	84	70 – 100

Table 2 Voids according to method EN 1097-4:2008 of dry compacted ordinary filler, PMMA/ATH composite dust and their mixtures

PMMA/ATH composite dust	Ordinary filler	Ordinary filler: PMMA/ATH composite dust = 5:1 wt.	Ordinary filler: PMMA/ATH composite dust = 8:1 wt.	Requirement according to SIST 1038-1
53 %	34 %	37 %	36 %	28% – 38%

2.2 PMMA/ATH composite dust as additive to bitumen

Silverson L5M homogenizer was used for mixing different quantities of PMMA/ATH composite dust in paving grade bitumen B 70/100 [5, 6]. To ensure a good dispersion dust was mixed in bitumen for 1.5 h at 170°C. Additionally we prepared sample containing 3 wt. % of paraffin wax and 25 wt. % of waste PMMA/ATH. Several test methods were used to evaluate quality of produced modified bitumen such as needle penetration at 25°C, softening point, Fraass breaking point and rut resistance potential (G*/sin(δ) (Tab. 3). With RTFOT ageing procedure also ageing potential of modified bitumen was evaluated.

Table 3 Properties of PMMA/ATH composite dust modified bitumen

PMMA/ATH composite dust content wt. %	Softening point [°C]	Fraass breaking point [°C]	Penetration at 25°C, 0.1 mm	G*/sin(δ) before ageing	G*/sin(δ) after RTFOT ageing
0	46.2	-11	78	1330	4790
25	53.6	-15	55	3650	14500
0 (3% wax)	71.0	-13	54		
25 (3%wax)	93.8	-12	34	6120	18900

With simple test methods such as needle penetration at 25°C, softening point and Fraass breaking point only insignificant differences between base bitumen and bitumen modified with PMMA/ATH composite dust were determined (Tab. 3). Only addition of paraffin wax significantly affected softening point. But both additives together seem to have multiplicative effect on softening point of bitumen.
From G*/sin(δ) measurements with dynamic shear rheometer we found significant differences between asphalts containing base bitumen and asphalts containing bitumen modified with PMMA/ATH composite dust. From these results increased resistance to permanent deformations was expected.

2.3 Asphalt mixtures containing PMMA/ATH composite dust

For laboratory testing four AC 8 asphalt mixtures were prepared. First mixture contained PMMA/ATH composite dust as additive to filler in mass ratio 1:5, second contained PMMA/ATH composite dust in paving grade bitumen B 70/100 in mass ratio 1:3, third was similar to second with additionally 3 wt. % paraffin wax and fourth reference was without PMMA/ATH composite dust [5, 7].
For all asphalt mixtures wheel tracking parameters and water sensitivity were determined. Wheel tracking tests were performed at 50 °C. Proportional rut depth of mixture containing PMMA/ATH composite dust in paving grade bitumen B 70/100 is approximately 3 times lower in comparison to the reference mixture (Fig. 1). The results of wheel tracking test are in good agreement with G*/sin(δ) values determined with binder test.
Increased water resistance of samples containing PMMA/ATH composite dust (ITS ratio) implies that waste PMMA/ATH particles in asphalt binder improve the adhesion performance between aggregate and bitumen (Tab. 4). From result it can be seen that more effective is addition of PMMA/ATH in bitumen.

Table 4 Properties of PMMA/ATH composite dust modified asphalt

Samples of AC 8 surf	ITS ratio at 25 °C [kPa]	ITS ratio at 25 °C [%]	Proportional rut depth at 50 °C [%]	WTSAIR at 50 °C
Reference mixture	907	93.1	18.3	0.46
PMMA/ATH composite dust added in filler	895	94.4	9.0	0.16
PMMA/ATH composite dust added in bitumen	1102	99.2	6.3	0.09
PMMA/ATH composite dust added in bitumen + 3% wax	1215	97.5	3.5	0.03

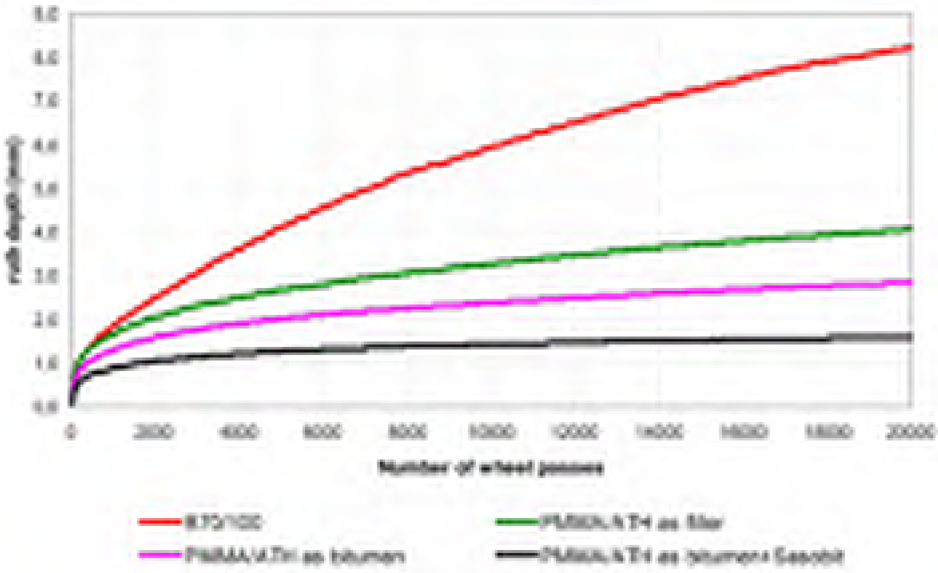

Figure 1 Results of wheel tracking test at 50 °C

For all asphalt mixtures compactability was determined and for three asphalt mixtures low temperature properties. Samples with PMMA/ATH composite dust added in filler give similar or even better results than reference mixture at these two tests. If PMMA/ATH composite dust is added in bitumen, than it is a bit harder to compact asphalt (compactability test gives higher result).

Table 5 Properties of PMMA/ATH composite dust modified asphalt

Samples of AC 8 surf	Compactability at 160 °C [kPa]	TSRST failure temperature [°C]	TSRST failure stress σ cry [MPa]	Tensile strength reserve, T Δβ, max. [°C]	Tensile strength reserve, Δβ, max. [MPa]
Reference mixture	26,4	-24,7	4,0	-8,4	3,8
PMMA/ATH composite dust added in filler	26,5	-28,1	4,4	-10,0	4,3
PMMA/ATH composite dust added in bitumen	30,4	-26,3	4,5	-6,3	4,4
PMMA/ATH composite dust added in bitumen + 3% wax	31,5				

3 Field experience

At the first two field trials we decided to use PMMA/ATH composite dust added in filler. In 2009 we produced 120 tons of asphalt containing PMMA/ATH composite dust and for comparison the same amount of ordinary asphalt. At construction of the test section with PMMA/ATH composite dust in binder course, workers noticed that in was easier to handle with asphalt containing PMMA/ATH composite. The driver of asphalt paver confirmed that workability of asphalt is improved, when it is containing PMMA/ATH composite dust. The test field with PMMA/ATH composite dust in wearing course is still monitored and there are no visible defects on the surface (fig 2).

a) b)

Figure 2 Construction of test field with PMMA/ATH composite dust in wearing course in 2009 (a.) and the same test field in 2013 (b).

Test field with PMMA/ATH composite dust added in bitumen was built in 2013. We produced 500 tons of asphalt containing PMMA/ATH composite dust. The production of bitumen containing PMMA/ATH composite dust was carried out in bitumen tank with mixer (fig. 3). We did not notice any problem when asphalt containing modified bitumen was produced. Also at construction of binder course with this asphalt was carried out smoothly.

Figure 3 Production of bitumen containing PMMA/ATH composite dust.

4 Conclusions

From laboratory tests performed on bitumen and asphalt we concluded that addition of PMMA/ATH composite dust always improves quality of asphalt. To our opinion the reason for improved properties of asphalt is in increased adhesion between bitumen and stone aggregates. As already known from previous studies addition of paraffin wax in asphalt improves resistance to permanent deformation. With this study we found out that both additives together have multiplicative effect on resistance to permanent deformation.
With test production and test fields we proved that addition of PMMA/ATH composite dust can be applied in normal batch type asphalt plant. On asphalt plant CGP in Drnovo we performed both dry and wet process. Test field from 2009 is a proof that PMMA/ATH modified asphalt is at least equally durable as ordinary asphalt. Altogether we can proclaim that PMMA/ATH modified asphalt is sustainable solution.

References

[1] Agostinacchio, M., Diomedi, M., Olita, S.: The use of marginal materials in road constructions: Proposal of an eco-compatible section, Advanced Testing and Characterization of Bituminous Materials –Loizos, Partl, Scarpas & Al-Qadi (eds), 2009 Taylor & Francis Group, London, ISBN 978-0-415-55854-9.

[2] Kennedy, T. W. and Ping, W. V.: Evaluation of Effectiveness of Antistripping Additives in Protecting Asphalt Mixtures from Moisture Damage, Journal of the Association of Asphalt Paving Technologists, from the Proceedings of Technical Sessions, 1991.

[3] Little, D., Epps, J.: The Benefits of Hydrated Lime in Hot Mix Asphalt, 2006, National Lime Association,.Cox, D. B. Integration of GPS with Inertial Navigation Systems. Global Positioning Systems and its Augmentations. 1998. Vol. I. P. 144-153. Available at: http://www.lime.org/BENEFITSHYDRATEDLIME2006.pdf

[4] Tušar, M., Beličič, A., Prešeren, M.: Asfalt z dodanim PMMA/ATH (Asphalt with added PMMA/ATH) Patent in Slovenia: SI23670 (A), 2012-09-28. Ljubljana: Urad Republike Slovenije za intelektualno lastnino (The Slovenian Intellectual Property Office), 2012.

[5] Tušar, M., Beličič, A., Prešeren, M., Šušteršič, E., Novič, M., Zupančič-Valant, A.: S PMMA/ATH kompozitnim prahom modificiran bitumen in uporaba le-tega (Bitumen modified with PMMA/ATH composite dust and its use) : Patent application in Slovenia: P-201200213, 2012-06-20. Ljubljana: Urad Republike Slovenije za intelektualno lastnino, (The Slovenian Intellectual Property Office) 2012.

[6] Šušteršič, E., Tušar, M., Zupančič-Valant, A.: Rheological and mechanical characterization of waste PMMA/ATH modified bitumen. Constr. build. mater., 2013, vol. 38, pages 119-125.

[7] Šušteršič, E., Tušar, M., Zupančič-Valant, A.: Effective concept of asphalt concrete modification with waste PMMA/ATH. Materials and Structures., 2013,

NEW SOLUTIONS FOR DISTRESSED PAVEMENT REHABILITATION OF VILNIUS CITY STREETS

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Abstract

Permanent deformation in asphalt pavement structures is one of the main pavement distress problems. The common asphalt pavement surface deformations are shoving and rutting at intersections, bus stops and bays, in heavy vehicle loaded urban streets due to acceleration, deceleration, slow moving or standing. It was determined that in many cases the failure was caused by asphalt layers fatigue and low resistance to shear flow. The laboratory research of high modulus asphalt concrete mixtures with different aggregate and binder types showed good results for city streets pavements performance improvement.

Keywords: *asphalt pavement, rutting, flow rutting, surface corrugation, plastic (permanent) deformation, high modulus asphalt concrete (HMAC)*

1 Introduction

Driving conditions and traffic safety depend on pavement roughness, which generally is a function of all distresses, and its severity level increases with pavement age. Asphalt pavement distresses may be classified into: surface defects (raveling, bleeding and polishing), deformations (rutting, shoving and corrugation), cracks (fatigue, thermal, longitudinal and slippage) and potholes. Although, the most common distresses are fatigue cracking, thermal cracking and rutting [1]. Recently, an increase in severity and extent of rutting in asphalt pavement structures in high traffic flow streets of Vilnius, bus lines and stops has been observed. Permanent deformations in city streets asphalt pavements cause the following concerns [2]: traffic safety, driving comfort, society reaction, and expenditures. For vehicles, there are reduced frictional characteristics (e.g., wheel path flushing), changing lanes becomes hazardous, and there is the risk of loss of control. Ruts influent steering accuracy and comfort can lead to accidents. Rut rehabilitation incurs costs, including user costs due to traffic flow interruptions and increased vehicle maintenance as well as rehabilitation costs. Permanent deformation in one or several pavement layers displays after sufficiently high amount of load repetition. Due to different structure materials properties pavement performs variously at particular loading and climatic conditions. Consequently, different rutting type may occur in asphalt concrete (AC) pavement rutting 2]–[5]: surface wear, initial densification, structural and flow rutting. The surface wear rutting forms only in the top layer of the asphalt, due to progressive loss of coated aggregate particles from the pavement surface, which is caused by combined environmental and tire influence [2], [3], [6]. This problem is not significant when usage of studded tires is controlled [2], [7]. The initial densification rutting forms during the first years of exploitation, due to insufficient compaction of AC layers or all structure layers [3]. The structural rutting affects all structure layers and is related with an appropriate pavement design (evaluation of load, weak subgrade, poor drainage, frost action, etc.), materials specification, and construction quality [2], [3]. Usually, structural rutting is a reflection of permanent deformation within granular base layers and subgrade. The flow rutting forms only in asphalt layers and is related with asphalt mixture mechanical properties, air voids content and mixture resistance to share flow. Flow rutting is the most common rutting type in EU [8]. The aim of this article is to analyze the practice and recommendations of other countries regarding pavement distresses (rutting) rehabilitation, to determine pavement conditions of the most distressed city streets of Vilnius and to suggest reliable solutions for pavement rehabilitation.

2 Permanent rutting rehabilitation solutions

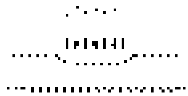
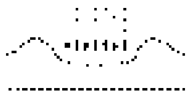
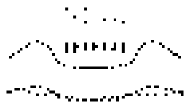
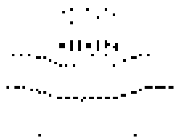
Permanent deformation (rutting) is very important because of its influence on vehicle movement (affecting vehicle tracking), safety (hydroplaning after rain) and dynamic loading (through surface profile variations) [3]. Rutted pavement rehabilitation demand is defined according to monitoring data through rut depth and severity level. The maximum rut depth in subgrade surface of national significance roads in Lithuania is 100-200 mm depending on maintenance level. The maximum rut depth in pavement surface is 40 mm [9]. However, rut depth and roughness is not restricted and there are no confirmed technical documentations for streets maintenance in city streets of Lithuania. Different agencies present different rutting severity levels (Table 1.).

Table 1 Rutting severity due to rut depth

Agency	**Rut depth [mm]**		
	Low severity level	**Medium severity level**	**High severity level**
NRC CNRC, Canada [2]	6 – 13	13 – 25	› 25
Transportation information center [4]	‹ 12,7	‹ 25,4	‹ 50,8
Washington State Department of Transportation	6,3-12,7	12,7-19,1	› 19,1
Ohio Department of Transportation	3,2-9,5	9,5-19,1	› 19,1
Texas Department of Transportation [10]	6,4-12,5	12,7-25,2	25,4-50,6
Illinois Department of Transportation [11]	‹ 4	4 – 9	› 9
DMRB, UK [12]	‹ 6	‹ 11	‹ 20

According to literature review, pavement degradation should be evaluated in further steps [2], [3], [7]: (1) visual inspection of the pavement; (2) transversal profile measurements; (3) pavement bearing capacity determination using falling weight deflectometer; (4) pavement structure materials sampling and testing (especially asphalt pavement layers and subgrade soils); (5) exploitation condition evaluation of pavement structure.
For simplicity, the rutting increment behavior can be observed from transversal profile analysis, which is useful for corrective actions selection [7]. According to Strategic Highway Research Program (SHRP) on Long-Term Pavement Performance program (LTPP) study pavement transverse profiles can provide information needed to select rehabilitation method such as shape and type of rutting, depth, and lateral location of longitudinal pavement deformations [13], [14]. On the basis of literature review and experience, asphalt pavement structure rehabilitation can be selected considering rutting profile type (Table 2.)

Table 2 Asphalt pavement structure rutting profile type [2], [7], [15]

Rutting type	Description	Transversal profile	Rehabilitation solutions
Wear rutting	Rutting is in the wearing layer, without flow deformations		Surface preservation (maintenance) · Surface treatment · Surface milling · Rut filling · Hot in-place recycling of wearing layer; · Wearing layer replacement.
Flow rutting of wearing layer	Rutting is in wearing layer with flow deformations		Surface preservation (maintenance) and improvement (repair) · Surface milling (short-term) · Hot in-place recycling of wearing layer; · Wearing layer replacement with stiffer AC mixture
Flow rutting of wearing and binder layers	Rutting is more than just in wearing layer with flow deformations		Pavement improvement (rehabilitation) · Asphalt pavement strengthening (replacing) with stiffer AC base or/and binder layer · Pavement strengthening (replacing) with PCC
Structural rutting	Deformations in all layers		Pavement structure improvement (reconstruction) · Improvement of pavement structure bearing capacity · Drainage improvement

Rehabilitation solution should be selected carefully considering all pavement design input data (traffic, subgrade type, drainage and environmental conditions, etc.) and life-cycle cost analysis. Medium and high severity non-structural rutting is repaired by improving the surface or pavement layers. If pavement structural bearing capacity during all seasons is sufficient, then surface layer rehabilitation solutions can be classified [2]: (A) surface preservation (maintenance) – surface milling and treatment extend pavement design life from 1 to 3 years; (B) surface improvement (repair) – wearing layer hot-in place recycling, wearing layer replacement (milling ‹ 50 mm AC layer thickness), AC overlay – from 3 to 10 years; (C) pavement improvement with AC (rehabilitation) – asphalt layers (milling › 125 mm thickness) and laying stiffer AC mixtures – from 15 to 20 years; (D) pavement improvement with Portland Cement Concrete (PCC) (rehabilitation) – ultra-thin concrete pavement can extend pavement design life from 5 to 15 years; conventional concrete pavement – from 15 to 30 years.

Rehabilitation using PCC overlay on an existing pavement surface is called white-topping (WT) [16], [17]. There are two types of WT: (1) bonded PCC Overlay – slab performs the same as existing pavement structure; (2) un-bonded PCC Overlay – slab performs as conventional PCC layer of pavement structure. According to Breyer [18], if pavement structure is frequently influenced by a specific loading, concrete pavement may be considered reasonable for low traffic flows. Semi-Flexible Pavement (SFP) is used in bus stops and intersection pavements in Finland, Sweden and Denmark. SFP consists of a porous asphalt pavement (voids between 25 percent and 30 percent) flooded with a high performance, micro-silica and Portland cement-based grout. SFP provides an alternative surfacing material tracked-vehicle roads, hardstands, and aircraft parking aprons [20]. Although, the biggest number of thermal cracking was observed in SFP after 5 years of exploitation in Road of Experimental Pavement Structures [21].

Rehabilitation using AC overlay should be done considering mixture properties and it's resistance to deformations. Choi [23] stated that shear resistance of a binder at high service temperatures was an important property for the flow rut resistance of asphalt and determined that dynamic viscosity can be considered as an indicator or rut resistance at low strains. Rese-

arch showed that rutting resistance of High Modulus Asphalt Concrete (HMAC) is twice higher than that of hot mix asphalt, and the fatigue resistance is 5–10 times higher [24]. Vaitkus and Vorobjovas [25] tested HMAC mixture properties and performance, which was made from lower quality aggregates, but used stiffer binders. Researchers determined that the lowest rut depth from Wheel Tracking Test (WTT) after 10000 cycles was obtained 0,77 mm in HMAC with crushed granite mineral aggregate and PMB 25/55-60, and RD was 3,5 times smaller than AC 16 AS (with PMB 45/80-55) which is often used to lay pavement structure in Lithuania.
Hot in-place recycling may be also applied for preservation and improvement of pavement surface layers. Recycling can be done at traffic line width and at deformed pavement part. The recycling technology adjusted for rut rehabilitation was developed in Lemminkainen, Finland and it is called Rut-Remix®. Rut-Remix technology is an economical solution for rehabilitation because it renews only deformed zone 1 m width. However, this technology can be applied only for wearing (low severity) rut, where rut depth is to 2-3 cm.

3 Experimental research

Research was done in sixteen distressed pavements of the city streets of Vilnius. Pavement transverse surface profile was measured in most damaged section of the street. Measurements were done using 3 m long straightedge. The distance from horizontal line to pavement surface was measured with ±1 mm accuracy. In order to determine deformation in asphalt layers, 150 mm diameter cores of pavement were taken in the measuring line. Each core was drilled with 20-30 mm covering. Drilled cores were tested in Road Research Laboratory of Road Research Institute of Vilnius Gediminas technical university. The thickness of every layer and the type of asphalt mixtures were determined. Summarized information, measured rut depth (RD) and severity level of distressed city streets pavements of Vilnius is shown in Table 3.

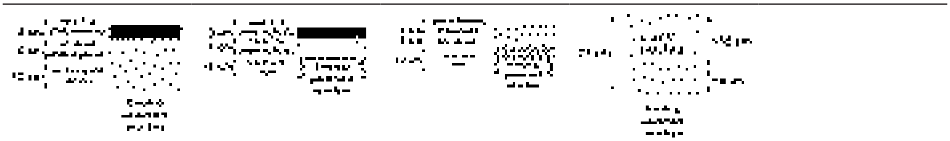
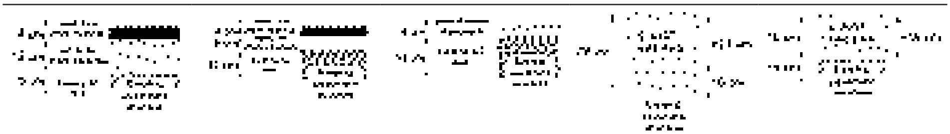
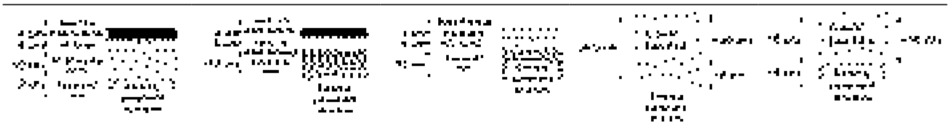
Table 3 Summarized information, measured RD and severity level of distressed city streets pavements of Vilnius

Section No.	Site characteristic	HVTF[1], v./d.	AC thickness, mm	Surface transvers slope direction	Measured RD, mm		Rutting type	Severity Level[2]
					Left	Right		
1.	intersection zone	3317	235	right	35-40	55-60	Structural and flow	High
2.	bus-stop section	580	150	left	35-40	30-35	Structural and flow	High
3.	intersection zone	264	150	right	20-25	30-35	Initial densification	High
4.	intersection zone	522	185	right	60-65	70-75	Structural	High
5.	uphill section	1309	195	right	15-20	15-20	Wear	Medium
6.	intersection zone	482	180	right	50-55	60-65	Flow	High
7.	intersection zone	206	190	right	30-35	30-35	Structural and flow	High
8.	intersection zone	771	110	left	50-55	30-35	Structural and flow	High
9.	bus-stop section	786	210	right	40-45	30-35	Flow	High
10.	intersection zone	657	260	right	60-65	50-55	Structural and flow	High
11.	intersection zone	482	195	right	10-15	10-15	Initial densification	Low
12.	intersection zone	482	160	right	65-70	60-65	Flow	High
13.	acceleration zone	1640	180	right	65-70	30-35	Wear and structural	High
14.	intersection zone	1152	250	left	60-65	20-25	Flow	High
15.	bus-stop section	318	180	left	80-100	80-100	Flow	High
16.	intersection zone	568	125	right	35-40	55-60	Structural and flow	High

[1] HVTF – Heavy vehicle traffic flow
[2] According to NRC CNRC regulations, see Table 1.

All types of permanent deformations were detected in distressed streets pavements: wear rutting (defined in No 5 and 13), initial densification (defined in No 3 and 11); structural rutting (defined in No 7) and flow rutting (defined in No 6, 9, 12, 14 and 15). It was also observed that in high severity flow rutting, deformation affected granular base layer, thus structural and flow rutting type was defined in No. 1, 2, 4, 8, 10 and 16. Structural and flow rutting established in most cases (56 %), flow rutting detected in fourth place (25 %), were noticed from distressed pavement transversal profiles and measurements.
Rehabilitation solutions for distressed pavements were carried out according to experimental research, KPT SDK 07 and RStO 12 pavement design guides, and Vaitkus and Vorobjovas' research [25]. Rut-Remix was proposed for pavement with low and medium rut severity preservation hot-in place recycle. Extra rehabilitation solutions, using rigid and semi-flexible pavements, were carried out for five sections (Table 4). Rehabilitation costs were defined individually in sections No. 4, 6, 9, 15 and 16 considering existing pavement milling, depose of waste, cracks sealing, quantity of materials, joint installations, laying works, compaction, joint sealing, curining. Costs were calculated with SISTELA software taking the prices from 2013. The costs for rehabilitation solutions are shown in Table 5. The comparison of rehabilitation solution costs is shown in Figure 1.

Table 4 Rehabilitation using AC, HMAC SF PCC and WT overlays

AC Overlay	HMAC Overlay	SF Overlay	PCC Overlay	WT Overlay
Section No. 4				
				
Section No. 6				
				
Section No. 9				
				
Section No. 15				
				
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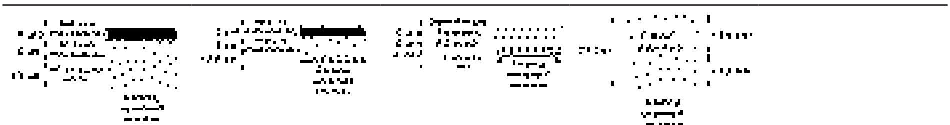				

Table 5 The cost for rehabilitation solutions costs

Section No.	Site characteristic		Rehabilitation cost (with VAT 21 %), €				
			AC Overlay	HMAC Overlay	SF Overlay	PCC Overlay	WT Overlay
4	Intersection zone	175 m²	17442,5	13994,6	12389,8	15268,5	12596,5
6	Intersection zone	217 m²	21466,3	12129,4	18446,9	18107,8	-
9	Bus-stop section	189 m²	16323,0	8892,7	13299,9	17222,5	13469,9
15	Bus-stop section	220 m²	18284,6	10224,3	20213,9	18242,3	15916,7
16	Intersection zone	420 m²	40974,0	23199,7	35652,2	35889,4	-

Rehabilitation cost using usual asphalt mixtures (AC) overlay determined higher comparing to other rehabilitation solutions in section No. 4, 6 and 16. Rehabilitation cost using SF overlay was about 8 % lower than PCC overlay cost. Rehabilitation using WT solutions could be used in sections No 6 and 16, because the existing asphalt layer thickness was too low. Although, rehabilitation cost of WT overlay was similar or slightly different from SF rehabilitation cost. HMAC overlay cost was 20-43 % lower than usual AC overlay cost and varies from 8 892.7 to 23 199.7 € depending on individual section condition. To sum up, rehabilitation using HMAC overlay was cheaper in most cases.

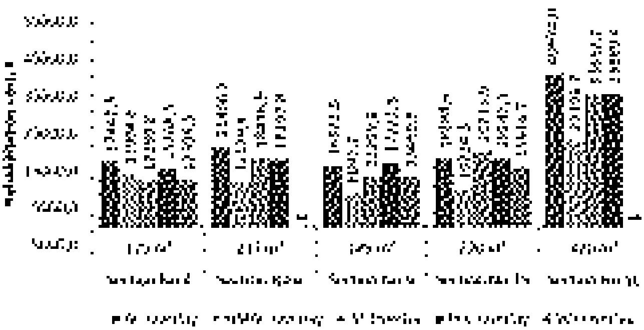

Figure 1 The comparison of rehabilitation solutions costs

4 Conclusions

The analysis of the distressed pavement structures has revealed that, rut depths exceed 60 mm and are deeper in 44 % of researched pavement sections. All types of rutting were detected in distressed streets pavements: wear rutting, initial densification (construction works failure), structural rutting (structure failure) and flow rutting (asphalt mixture failure). However, deformations mostly emerged in asphalt wearing or/and asphalt binder layers due to insufficient asphalt mixture shear flow (flow rutting resistance).
In most cases rehabilitation cost for solutions with high modulus asphalt concrete (HMAC) layers was 20-43 % cheaper than asphalt concrete (AC) overlay rehabilitation cost.
Semi-flexible (SF) pavement overlay cost is less or more similar to concrete withe-topping overlay cost and 8 % lower than Portland cement concrete (PCC) overlay, and could be used when there is need of pavement bearing capacity improvement.
To conclude, the experimental research has shown that, rehabilitation solution should be derived individually, although high modulus asphalt concrete overlay was a reasonable solution for pavements where flow rutting occurs in binder layer. Hot in place recycling is a very relevant solution for less distressed pavements (wearing rutting, surface course rutting) rehabilitation.

References

[1] Huang, Y. H.: Pavement Analysis and Design, Second edition, New Jersey, 2004.

[2] NRC CNRC: Rut Mitigation Techniques at Intersections. Canada, 2003.

[3] Morosiuk, G. & Riley, M.: Volume 6. Modelling Road Deterioration and Works Effects in HDM-4, in Highway development & Management, World Road Association, 2004.

[4] Walker, D. Entine, L. & Kummer, S.: Pavement Surface Evaluation and Rutting. Asphalt Roads, Transportation Information Center University of Wisconsin-Madison, 2002.

[5] NCHRP 1-37A: Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures, Champaign, Illinois, USA, 2004.

[6] Oscarsson, E.: Evaluation of the Mechanistic–Empirical Pavement Design Guide Model for Permanent Deformations in Asphalt Concrete, Road Materials and Pavement Design, Vol. 12, No. 1, pp. 37–56, 2011.

[7] NCHRP Report.468: Contributions of Pavement Structural Layers to Rutting of Hot Mix Asphalt Pavements, Washington, USA, 2002.

[8] COST 333: Development of New Bituminous Pavement Design Method, Brussels, Luxembourg, 1999.

[9] LAKD: Kelių priežiūros vadovas Part I. KPV PN-05., Vilnius, Lithuania, 2005.

[10] Texas Department of Transportation: Pavement Design Guide, USA, 2011.

[11] IDOT: Bureau of Design and Environment Manual. Pavement rehabilitation. Ilinois, USA, 2012.

[12] DMRB: Data for Pavement Assessment HD 29/08, London, UK, 2008.

[13] Fang, H., Haddock, J. E., White, T. D. & Hand, A. J.: On the characterization of flexible pavement rutting using creep model-based finite element analysis, Finite Elements in Analysis and Design, Vol. 41, No. 1, pp. 49–73, 2004.

[14] Gramling, W. L., Hunt, J. E. & Suzuki, G. S.: Rational Approach to Cross-Profile and Rut Depth Analysis, Transportation Research Board, National Research Council, Washington D.C., pp. 173–179, 1991.

[15] Vaitkus, A., Čygas D. & Kleizienė, R.: Research of asphalt pavement rutting in Vilnius city streets, in The 9th International Conference "Environmental Engineering, 2014.

[16] Harrington, D., et al.: Guide to Concrete Overlays. Sustainable Solutions for Resurfacing and Rehabilitating Existing Pavements, Iowa, USA, 2008.

[17] Kleizienė, R., Vaitkus, A. & Čygas, D.: The Analysis Of Concrete Pavement Application Possibilities For Road Pavement Structures, in The XXVIII International Baltic Road Conference, 2012.

[18] Breyer, G.: Entscheidungskriterien für den Bau von Betonfahrbahndecken in Österreich, in BETONOVÉ VOZOVKY 2004 Sborník přednášek 1. mezinárodní konference, 2004.

[19] Gawedzinski, M. J.: Evaluation of semi-flexible (resin-modified) pavement report, Illinois, USA, 2008.

[20] Ahlrich R. C. & Anderton, G. L.: Construction and Evaluation of Resin Modified Pavement, 1991.

[21] Vaitkus, A., et. all.: The Road of Experimental Pavement Structures: Experience of Five Years Operation, The Baltic Journal of Road and Bridge Engineering, Vol. 7, No. 3, pp. 220–227, 2012.

[22] Tarefder, R. A., Zaman, M. & Hobson, K.: A Laboratory and Statistical Evaluation of Factors Affecting Rutting, The International Journal of Pavement Engineering, Vol. 4, No. 1, pp. 59–68, 2003.

[23] Choi, Y. K.: Visco-Elastic Analysis of the Elastomeric Binder Shear Resistance in Relation to Asphalt Rutting, Road Materials and Pavement Design, Vol. 12, No. 4, pp. 767–794, 2011.

[24] Lee, H. J., Lee, J. H. & Park, H. M.: Performance evaluation of high modulus asphalt mixtures for long life asphalt pavements, Construction and Building Materials, Vol. 21, No. 5, pp. 1079–1087, 2007.

[25] Vaitkus A. & Vorobjovas, V.: Use of local aggregates in high modulus asphalt concrete layers, Građevinar, Vol. 65, No. 4, pp. 353–360, 2013.

THE IMPACT OF COMPACTION ENERGY ON THE PROPERTIES OF ASPHALT LAYERS

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Abstract

At an installation of the hot mix asphalt on a construction site, it is necessary to achieve the designed quality of the built asphalt layer. On the quality of the built asphalt layer affect the temperature of the asphalt mix, the quality of a bearing layer, external weather conditions and energy invested during the installation of hot mix asphalt. Laboratory tests aimed to determine the dependence of the invested variable compaction energy on asphalt mix AC 8 surf 50/70 AG1 M4 and the achieved density of the test specimens in the laboratory conditions. The designed asphalt mix was made up in the laboratory of stone fractions of igneous origin, stone dust of dolomite origin and bitumen 50/70. The obtained results indicate that the invested compaction energy significantly affects on the achieved quality of the laboratory made asphalt specimens.

Keywords: hot mix asphalt, quality of the built asphalt layer, compaction energy.

1 Introduction

Asphalt layers are the elements of a pavement structure that take traffic load and pass it on the bearing layer structures, and are made by an installation of hot asphalt mixes, a three-component or multi-component composite [1] of the composition defined by volume shares of grained mineral mixture, bitumen and air in the voids. Asphalt mixes should contain 3-5% of voids [2], and any further deviation leads to the appearance of permanent deformations. Voids are the most usually measured volumetric feature of hot asphalt mixes because their share in the total asphalt mix affects on the stability and persistence of the same [3-6]. The share of voids (air) of 5-8%, or less than 3%, results in the formation of cracks and the emergence of tracks on asphalt pavements [7-8] and the inability of spreading asphalt and additional densification due to the traffic load. When installing asphalt mix, it is necessary to achieve the designed densities and porosity reduction in the resulting layer because any increase of voids by 1% (more than 6-7%) leads to a 10% reduction in the designed durability of asphalt pavement [9]. The density of asphalt layers is one of the most important factors that affects the durability of the asphalt pavement. Compaction of hot mix asphalt reduces the proportion of voids (air) and increases the unit weight of the compacted mix. According to studies [10-16], compaction is probably the most important activity in the construction of each of the layers of the pavement structure, and especially of asphalt layers. Well compacted embedded asphalt layers provide some waterproofing, strength and resistance to the emergence of tracks and prevent excessive aging, i.e. oxidation of bitumen. Workability of the asphalt mix depends on the composition of bituminous mix, the quality of bitumen and its viscosity, as well as on the temperature at which the installation is done. Hot asphalt mixes are designed by classical methods (Marshall, Hveem, Duriez or Approximate) and by methods of recent approaches (the Superpave method and harmonized European standards). Compaction of laboratory specimens of bituminous mixes is done by compactors, such as the Marshall compactor, the Hveem stabilometer or the Superpave gyratory compactor. This study applied the Marshall method of compacting designed hot mix asphalt. Designing hot mix asphalt by the Marshall method is linked to the thirties of the last century, the author of which is Bruce Marshall. They were adopted with the aim of designing an optimal gradation of aggregates, fillers and bituminous binder for asphalt pavement curtains resistant to permanent deformations. Today, the Marshall method is used in 38 countries around the world [19]. Asphalt mixes designed for installation into pavement structures for higher traffic loads contain a greater proportion of larger mineral fractions and during the installation require greater invested compaction energy. Mixes that are built into the structures of lower traffic loads contain a greater proportion of fractions of finer granulation, and are thus easily compacted. The American standard AASHTO (American Association of State Highway and Transportation Officials Standards) prescribes compaction criteria for asphalt mix specimens depending on traffic load of the pavement structure (Table 1), while the Croatian and European standards prescribe compaction criteria by the Marshall compactor based on 50 strokes on each side of the asphalt mix specimen [20].

Table 1 The Marshall method designed criteria (Asphalt Institute, Lexington, KY,1979.) [19]

Criterion	**Low traffic $<10^4$ ESALs**		**Medium traffic 10^4-10^6 ESALs**		**Heavy traffic $>10^6$ ESALs**	
	Min	Max	Min	Max	Min	Max
Compaction (number of strokes on each side of the specimen)	35		50		75	
Stability (min)	2 224 N		3 336 N		6 672 N	
% Voids (air)	3	5	3	5	3	5
ESALs – equivalent single axle load (equivalent to single-shaft load)						

The problem of density of asphalt mix before and after compaction is the subject of many studies. Neubauer [21] studied the effect of compaction temperature of hot mix asphalt onto volume changes, and came to the conclusion that the density decreases from the centre to the surface, i.e. to the bottom or the top of the asphalt specimens in the mold. Raab, in her researches, [22] found an unbalanced distribution of voids (air) immediately after compaction, where the asphalt specimen had a lower content of voids (air) in the middle and the upper surface than on the edges (edge lines) and in the lower parts. Since, during compaction, bitumen in the mixture acts as a lubricant, and with its share leads to a reduction of friction between the grains of the mineral mix, these are the investigated and the quantitative relationships of the compaction level and the voids filled with bituminous binder, wherein the results obtained indicate that the reduction of voids filled with bitumen binder for 4-12 % will result in the reduction of stiffness modulus by 55 % [23]. In this paper, the authors examined in the laboratory conditions the impact of variable compaction energy, at constant compaction temperature, of asphalt specimens of hot mix asphalt AC 8 surf 50/70 AG1 M4.

2 Experimental part

Laboratory tests include making hot mix asphalt AC 8 surf 50/70 AG1 M4 prepared under controlled conditions. Asphalt is the mixture of a fixed gradation of mineral mixture, the share of bitumen and the temperature at which compaction of test specimens is done (155°C). In order to determine in the laboratory the impact of the changing compaction energy on the properties of the test specimens of asphalt samples, it is carried out the preparation of specimens exposed to a variable number of strokes in the Marshall compactor. In the paving laboratory, the properties of asphalt mixtures specimens are tested according to HRN EN 13108-1 [24] made from stone fractions 0/2, 2/4 and 4/8 mm of volcanic origin from the Vetovo quarry, stone dust category label KB-I from the Veličanka quarry and road bitumen 50/70, MOL producer, the Republic of Hungary.

2.1 Composition of designed asphalt mix

Designed asphalt mixture contains in its composition stone fractions from the Vetovo quarry (an eruptive material of silicate composition) and stone dust from the Veličanka quarry (carbonate composition sediment). Table 2. shows the shares of certain fractions in the designed mineral mixture and the average densities determined in accordance with the standards HRN EN 1097-7 and HRN EN 1097-6 [25, 26].
Table 3. shows the gradation of the constituent mineral fractions determined according to the standard HRN EN 12697-2 [27].

Table 2 Densities and shares of stone fractions in the overall mineral mixture

Stone materials	**Fraction marks**	**Share in the mineral mixture [%(m/m)]**	**Density [t/m³]**
"Veličanka"	KB	7.4	2.8
"Vetovo"	0/2	31.2	2.88
"Vetovo"	2/4	23.8	2.9
"Vetovo"	4/8	37.6	2.8

Table 3 Gradation of used mineral fractions and filler

Opening of the sieve [mm]	**Passage through the sieve [%(m/m)]**			
	Stone dust	**Vetovo quarry**	**Vetovo quarry**	**Vetovo quarry**
		0-2	**2-4**	**4-8**
0.063	70.1	4.1	1.8	1.3
0.25	100	20.5	2.4	1.7
1	100	64.0	3.7	1.9
2	100	95.6	12.1	2.1
4	100	100	92.7	7.5
8	100	100	100	94.6
11.2	100	100	100	100
16	100	100	100	100
22.4	100	100	100	100
31.5	100	100	100	100

Used binder is a road bitumen 50/70, by MOL producer, from the Republic of Hungary. Standard properties of the road bitumen include density ρ_B = 1.012 g/cm^3, the softening point PK = 50.3°C and penetration PEN = 57 1/10 mm, tested according to the standards HRN EN 1425:2012 and HRN EN 1426:2008 [28, 29].

2.2 Designing the composition of asphalt mix

Gradation of the designed asphalt mixture is shown in Figure 1., while the soluble proportion of road bitumen is tested according to HRN EN 12697-1 [30] and adopted in the amount of 5.9%.

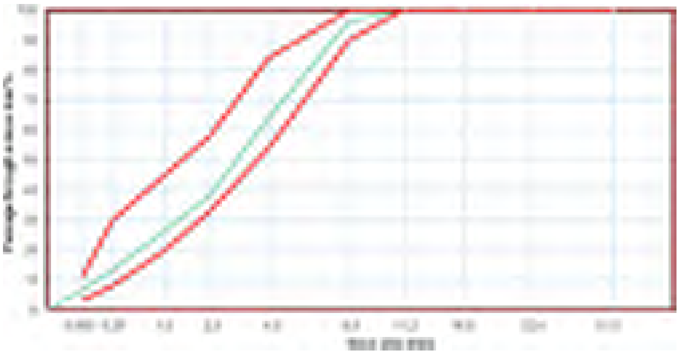

Figure 1 Gradation of the designed asphalt mixture

For each group of asphalt mix, the three test specimens were made according to the standard compaction procedure on the Marshall compactor with the falling hammer. The specimens, according to Annex C of the standard HRN EN 13108-20 [20], are compacted under variable number of strokes. Table 4. shows the conditions of preparation of test specimens of asphalt specimens. The compaction of the asphalt mixture is done by 1-70 strokes (on each side), where the number of strokes subsequently increases by 10 for each subsequent group (Group 2-8). Figure 2. shows the specimens compacted with 10, 30 and 70 strokes, i.e. the samples of the Group 2, Group 4 and Group 8.

Table 4 Conditions for preparing asphalt specimens

Test sizes	Unit measures	Gr. 1.	Gr. 2.	Gr. 3.	Gr. 4.	Gr. 5.	Gr. 6.	Gr. 7.	Gr. 8.
Hammer mass	g	4.560	4.560	4.560	4.560	4.560	4.560	4.560	4.560
Height of falling hammer	mm	460	460	460	460	460	460	460	460
Number of layers	–	1	1	1	1	1	1	1	1
Number of strokes per layer	–	2×1	2×10	2×20	2×30	2×40	2×50	2×60	2×70
The diameter of the specimens after compaction	mm	101.2	101.5	101.6	101.4	101.5	101.6	101.5	101.4
The height of the sample aft. compaction	mm	80.3	68.3	65.4	64.3	63.0	62.5	61.9	61.9

Figure 2 Laboratory asphalt specimens

After cooling asphalt specimens, the following properties were determined on the same:
- the density of the asphalt specimens according to the standard HRN EN 12697-6 [31],
- the density of asphalt mixture according to the standard HRN EN 12697-5 [32],
- the share of voids according to the standard HRN EN 12697-8 [33],
- voids filled with bitumen according to the standard HRN EN 12697-8 [33].

2.3 Achieved results

Table 5. presents the results of the performed laboratory tests. From Table 5., it is evident that the densities of asphalt specimens fall significantly with the decrease in the number of strokes at which compaction is done. The reduction of the number of strokes during compaction leads to the growth of share of voids in the test specimens.

Table 5 The results achieved after compaction in the Marshall compactor

Group	Density [kg/m³]		Share of voids [%]	The thickness of the specimen [mm]	The diameter of the mold [mm]
	Marshall	Asphalt mixture			
Group 1.	2.120	2.566	17.4	80.3	101.2
Group 2.	2.316	2.566	9.7	68.3	101.5
Group 3.	2.397	2.566	6.6	65.4	101.6
Group 4.	2.433	2.566	5.2	64.3	101.4
Group 5.	2.487	2.566	3.1	63.0	101.5
Group 6.	2.497	2.566	2.7	62.5	101.6
Group 7.	2.509	2.566	2.2	61.9	101.5
Group 8.	2.511	2.566	2.1	61.9	101.4

3 The dependence of the results achieved

The further study shows the dependences of variable compaction energy in the Marshal compactor and generated voids in asphalt specimens, the altitude of test specimens, as well as the densities of asphalt specimens.

3.1 Dependence of voids and compaction energy

Figure 3. shows the ratio of voids in asphalt specimens and variable compaction energy in the Marshall compactor. By applying the obtained values onto the coordinate axes, it can be seen exponential relationship between the observed values and, consequently, is analyzed the link model which is as follows:

$$U=ae^{bx} \tag{1}$$

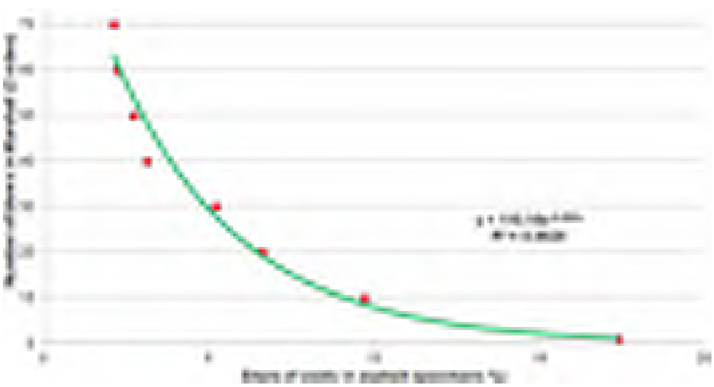

Figure 3 The ratio of the voids and the variable compaction energy

Figure 3. shows a very high coefficient of determination, $R^2 = 0.9928$, which indicates to the dependence of realized values of share of voids and effects of variable compaction energy in laboratory conditions.

3.2 Dependence of variable compaction energy and the altitude of asphalt specimens

Figure 4. shows the dependence of variable compaction energy and the altitude of asphalt specimens. The values obtained are applied on the axes of the coordinate system and is analyzed the link model which is as follows:

$$V=ax^b \quad (2)$$

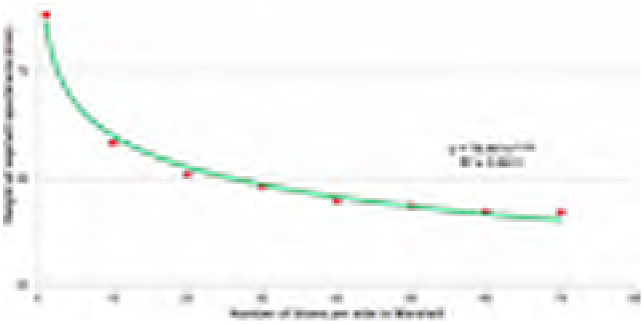

Figure 4 The ratio of compaction energy and the altitude of asphalt specimens

In Figure 4., it is evident that due to the growth of compaction energy, there follows a decrease in the altitude of test specimens and higher densities. It is observed a very high coefficient of determination, $R^2 = 0.9911$, which indicates that the mathematical notation of the referred dependence realistically describes the observed values.

3.3 Dependence of variable energy and densities of test specimens

Figure 5. shows the dependence of densities of asphalt specimens and variable compaction energy in laboratory conditions. The values obtained are applied on the axes of the coordinate system and is analyzed the following mathematical model:

$$G_{AU}=ax^b \quad (3)$$

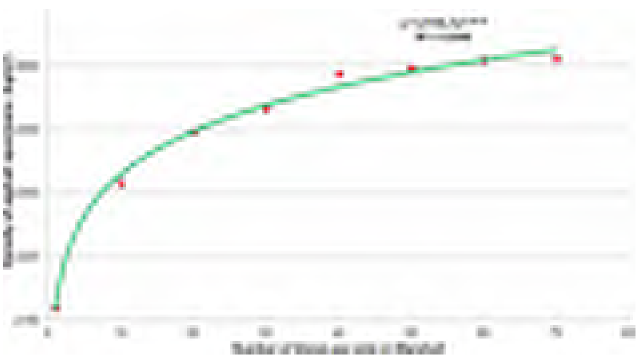

Figure 5 The ratio of variable energy and densities of asphalt specimens

The shown dependence in Figure 5. suggests to the conclusion that the increase of the invested compaction energy leads to higher densities of asphalt specimens. The coefficient of determination, R^2, of the displayed dependence amounts to 0.994 and indicates that the mathematical notation realistically describes the observed properties. Correlations of the observed properties as well as forms and the strengths of bonds for obtained results are shown in Table 6.

Table 6 Correlations of properties

Property of test specimen	Variable	Link form	Coefficients		The coefficient of determination
			a	b	R^2
Voids and energy compaction	(x) – voids	$U=ae^{bx}$	110.16	-0.265	0.9928
Energy and altitude of test specimen	(x) – the number of strokes in Marshall compactor	$V=ax^b$	79.641	-0.062	0.9911
Energy and densities	(x) – the number of strokes in Marshall compactor	$G_{AU}=ax^b$	2,116.7	0.0415	0.994

4 Conclusion

The quality of the built asphalt layer is significantly affected by invested compaction energy during installation. It is necessary to perform the installation of bituminous mixture at the designed temperature in dependence relation to the composition of the asphalt mixture. The density of the asphalt layer is defined as the weight of the mixture in a given volume. In the process of compaction, the volume of the layer reduces which results in the reduction of share of voids in the asphalt layer. Initial compaction has a far greater impact on the proportion of share of voids in relation to a further increase in compaction energy. It is also evident that the altitude of test specimens is reduced by 22.9% between the specimens compacted with 2x70 strokes (Group 8) and the specimens compacted with 2x1 strokes (Group 1). The differences in densities of the test specimens between the Group 8 and the Group 1 were 18.4%. In conclusion, the timely compaction and invested energy both prolong the durability of the built asphalt layer with minimal occurrence of deformations during the designed period of exploitation.

References

[1] The American Asphalt Institute, Mix Design Methods for Asphalt Concrete and Other Hot-mix Types, 3rd Edition (1963)

[2] Roberts, F.L., Kandhal, P.S., Brown, E.R., Lee, D.Y., and Kennedy, T.W. (1991). Hot Mix Asphalt Materials, Mixture Design, and Construction. National Asphalt Paving Association Education Foundation. Lanham, MD.

[3] Roberts, F.L., Martin, A.E.: Hot Mix Asphalt Materials, Mixture Design and Construction, NAPA Education Foundation, Lanham, Maryland, Second Edition (1996)

[4] Huber, G. A., and Herman, G.H. (1987). Effect of Asphalt Concrete Parameters on Rutting Performance: A Field Investigation. Association of Asphalt Paving Technologists, vol. 56: pp.33-61.

[5] Ford, Miller. Pavement Densification Related to Asphalt Mix Characteristics. Paper presented at 67th annual Transportation Research Board, January 1988.

[6] Brown, E. R., and Cross, S. (1989). A Study of In-Place Rutting of Asphalt Pavements. Association of Asphalt Paving Technologists, vol. 58.

[7] Roberts, F. L., Kandhal, P.S., Brown, E.R., Lee, D.Y., Kennedy, T.W. (1996). Hot Mix Asphalt Materials, Mixture Design, and Construction. NAPA Research and Education Foundation. Maryland, USA.

[8] Berthelot, C. F. (2003). Pavement Materials and Design. Course Material for CE 417. University of Saskatchewan.

[9] Linden, R.N.; Mahoney, J.P. and Jackson, N.C. (1989). The Effect of Compaction on Asphalt Concrete Performance. 1989 Annual Meeting of the Transportation Research Board, Washington, D.C.

[10] Shell,1990: The Shell bitumen Handbook. Shell, Chertsey (UK).

[11] Scherocman, J.A. and Martenson, E.D. (1984). Placement of Asphalt Concrete Mixtures. Placement and Compaction of Asphalt Mixtures, F.T. Wagner, Ed. ASTM Special Technical Publication 829. American Society for Testing and Materials. Philadelphia, PA. pp. 3-27.

[12] Scherocman, J.A. (1984, March). Guidelines for Compacting Asphalt Concrete Pavement. Better Roads, Vol. 54, No. 3. pp. 12-17.

[13] Geller, M., 1984. Compaction Equipment for Asphalt Mixes. Placement and Compaction of Asphalt Mixtures, American Society for Testing and Materials, Special Technical Publication 829, pp. 28-47. Wagner, Philadelphia.

[14] Brown, E.R. (1984). Experiences of Corps of Engineers in Compaction of Hot Asphalt Mixtures. Placement and Compaction of Asphalt Mixtures, F.T. Wagner, Ed. ASTM Special Technical Publication 829. American Society for Testing and Materials. Philadelphia, PA. pp. 67-79.

[15] Bell, C. A. Hicks, R.G. and Wilson, J.E. (1984). Effect of Percent Compaction on Asphalt Mixture Life. Placement and Compaction of Asphalt Mixtures, F.T. Wagner, Ed. ASTM Special Technical Publication 829. American Society for Testing and Materials. Philadelphia, PA. pp. 107-130.

[16] Hughes, C.S.: Importance of Asphalt Compaction. Better Roads, Vol. 54, No. 10. pp. 22-24, 1984.

[17] Sousa, J. B., Way, G., Harvey, J.T., Hines, M. (1995). Comparison of Mix Design Concepts. Transportation Research Board. Washington, D.C., n1492: pp.151-160.

[18] Asphalt-Institute (1997). Mix Design Methods for Asphalt Concrete and Other Hot- Mix Types.

[19] www.pavementinteractive.org/article/marshall-mix-design/seen. 09.01.2014.

[20] HRN EN 13108-20:2007/Ispr.1 Bituminous mixtures – Material specifications – Part 20: Type Testing (EN 13108-20:2006/AC:2008)

[21] Neubauer, O., “Study of Selected Parameters Relevant for Gyratory Compaction”, International Workshop on the Use of the Gyratory Shear Compactor, LCPC, Nantes, 12-13 December, (1996).

[22] Raab, C., González Lakehal, S. and Būdvytytė, G., “Ausgewählte Aspekte der Belagsprüfung”, Strasse und Verkehr, Nr10, pp 377-382, October, (2000).

[23] Powell, W.D. and D. Leech, (1983) Compaction of bituminous road materials using vibratory rollers. TRRL Laboratory Report 1102, Crowthorne, (UK)

[24] HRN EN 13108-1:2007 Bituminous mixtures – Material specifications – 1. part: Asphalt Concrete (EN 108-1:2006)

[25] HRN EN 1097-7:1999 Testing of mechanical and physical properties of aggregates – Part 7: Determination of the density of filler – pycnometry method (EN 1097-7:1999)

[26] HRN EN 1097-6:2000 Testing of mechanical and physical properties of aggregates – Part 7: Determination of density and water absorption (EN 1097-6:2000+AC:2002)

[27] HRN EN 12697-2:2003 Bituminous mixtures – Test methods for hot mix asphalt – 2nd Part: Determination of granulometric composition

[28] HRN EN 1425:2012 Bitumen and bituminous binders – Characterization of visible properties (EN 1425:2012)

[29] HRN EN 1426:2008 Bitumen and bituminous binders – Determination of penetration by needle (EN 1426:2007)

[30] HRN EN 12697-1:2003 Bituminous mixtures – Test methods for asphalt produced by hot process – 1.part: Soluble share of binder

[31] HRN EN 12697-6:2003 Bituminous mixtures – Test methods for asphalt produced by hot process – 6th Part: Determination of density of asphalt samples

[32] HRN EN 12697-5:2003 Bituminous mixtures – Test methods for asphalt produced by hot process – 5. Part: Determination of density of asphalt mixtures

[33] HRN EN 12697-8:2003 Bituminous mixtures – Test methods for asphalt produced by hot process – 8. Part: Determination of voids in asphalt samples

INDIRECT TENSILE TEST OF ASPHALT MIXTURE STIFFNESS MODULUS

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Abstract

Mechanistic – empirical calculation method is applied worldwide in calculation of new pavement structures and evaluation of the existing ones. The majority of mechanistic methods of dimensioning is based on the evaluation of the structural behaviour of structures, i.e. critical relation of stress and deformity during application of certain load on the pavement. Stiffness of asphalt mixtures is an essential parameter in evaluation of induced load, temperature stress and distribution of deformities in pavement structure. In certain parts of Croatia the average maximal summer temperatures on pavement surface reach approximately 40°C. At such high temperatures, the asphalt stiffness modulus is significantly decreased and there is a high possibility of occurrence of driving surface plastic flow in the form of rutting. During winter, temperatures decrease to -10°C and the combination of low temperatures and cyclic changes, with very high stiffness modulus, may cause cracks in asphalt pavement. Systematic measurement of asphalt mixture stiffness modulus was carried out within the research through application of the indirect tensile test method at temperatures those mixtures would be exposed to during their project life span. This Paper presents elaboration of stiffness modulus test results achieved through application of the indirect tensile test on several types of asphalt mixtures at different temperatures. Furthermore, it presents the stiffness modulus test results for several asphalt concrete mixtures AC 11 of different composition, i.e. of different physical – mechanical properties.

Keywords: stiffness modulus, asphalt mixture, indirect tensile test, rutting

1 Introduction

The goal of this research was to determine the stiffness modulus at mixture exploitation temperatures for several types of hot asphalt mixtures used today for construction of roads in Croatia, with the purpose of obtaining input data for control of stresses and strains occurring in pavement structure. Furthermore, another purpose was to establish how the mixture composition effects variation of stiffness modulus.

Until this research, obsolete and imprecise data on material properties were used in Croatia, obtained based on testing of dynamic modulus of elasticity in dependence on temperature, conducted almost 50 years ago [1] (Dormon and Edwards, 1967).

The effect of temperature variation to the stiffness modulus of seven different asphalt mixtures was determined within the first part of testing, resulting in curves representing variation of stiffness modulus of those mixtures due to temperature variation. The testing temperatures were within the limits of temperature range occurring on pavements in Croatia during the course of one year. The effect of asphalt-concrete AC 11 mixture variation to the stiffness was tested in the second part of the testing.

2 Indirect tensile test

Asphalt mixture stiffness modulus may be measured in several ways, according to the standard HRN EN 12697-26 [2]. The indirect tensile test was selected (Appendix C of the abovementioned standard), due to the simplicity and quickness of measurement, as well as preparation of specimen (in comparison to other measurement methods defined in the standard) and due to the availability of measurement equipment.
The majority of asphalt mixtures is not elastic but goes through some form of plastic deformity after application of load. However, if the load is low in comparison to the strength of material and if it is applied repeatedly, the deformity is almost completely reversible after each repetition.
Resilient or reversible modulus is the ratio of repeated stress and reversible (resilient) relative strain, due to the brief repeated load application which corresponds to the load from wheels of moving vehicles. Resilient modulus describes the efficiency of different pavement structure courses to distribute the stresses, resulting from traffic load, within the pavement structure. It is used in the theory of elasticity for forecasting of reversible relative deformity or displacement during mechanistic projecting of flexible pavement structures.
In laboratory the resilient modulus of asphalt mixtures is most frequently determined through the indirect tensile test (HRN EN 12697-26, Appendix C). The testing is non-destructive since the stresses are very low.
The indirect tensile test is used for measurement of small deformities of asphalt specimens. It is a non-homogenous testing, with assumption that the tested material is isotropic material with constant Poisson ratio. It is conducted by application of repeating impulse compressive load on two diametrically opposed „strips" of a cylindrical specimen. The central part of the specimen is then submitted to predominantly constant tensile stress in diametrical plane which connects the „load strips". This results in reversible deformity in horizontal diametrical plane.
In case the vertical variation of diameter is not measured, the assumed value of the Poisson coefficient for all asphalt materials is 0.35 for all temperatures [2]. Due to the viscoelastic properties of asphalt mixtures, duration of load application effects significantly the value of stiffness modulus and the goal is to achieve as realistic as possible simulation of actual conditions of load. The stress in pavement is maximal at the point above which the load is applied while it decreases with the distance from this point. Due to that, the assumed load pulse during testing is a triangular, square or sinusoidal function. According to standard [2] this procedure is classified under tests with sinusoidal load. It is recommended that the shape of load curve is approximately a wave with load shape factor 0.6. Since the pulse tests were developed for „inspection" measurements, only duration and the maximal applied value of repeated load are controlled during testing and not the shape of the wave. However, the shape of the load curve is recorded during measurement and the obtained stiffness modulus is corrected subsequently through an equation to the pre-defined load shape factor 0.6.

3 Testing programme

All the tests were conducted from the year of 2010 to 2012 in the Laboratory for asphalt and road soil mechanics, Institut IGH d.d. Zagreb. Asphalt mixture specimens were sampled on several actual construction sites, in compliance with the standard (HRN EN12697-27). Afterword, the specimens were prepared according to the standard HRN EN12697-28 and the testing of physical – mechanical properties of asphalt mixtures was conducted on them. The following was determined: soluble share of binder (HRN EN12697-1), grain size distribution (HRN EN12697-2) and thickness of asphalt mixture (HRN EN12697-5). Specimens of diameter of Φ100 mm and height of 60-65 mm were compacted in the laboratory according to the standard Marshall procedure (HRN EN12697-30). Asphalt specimen dimensions were determined

according to the standard HRN EN 12697-29, the density of asphalt specimens according to HRN EN 12697-6 and voids in asphalt specimens according to HRN EN 12697-8; the Marshall testing was carried out according to the standard HRN EN 12697-34.
During the first part of testing 848 measurements of the stiffness modulus were carried out, at four different temperatures and on 106 specimens prepared from seven asphalt mixtures [3]. All the specimens were tested at four temperatures: +5°C, +15°C, +25°C and +35°C, with respect to the asphalt pavement temperature range in Croatia (-8°C to +38°C,) and with respect to the equipment manufacturer's recommendation (+0°C to +35°C). Four temperature measurements were selected as the optimum number for obtaining the curve of the stiffness modulus dependence on temperature. The specimens were tested in both directions and, in case the values of measured modulus did not show too large deviations, their mean value is the specimen stiffness modulus at a certain temperature. Table 1. presents the first part of the asphalt mixtures stiffness modulus testing programme.

Table 1 Stiffness modulus testing programme (first part of testing)

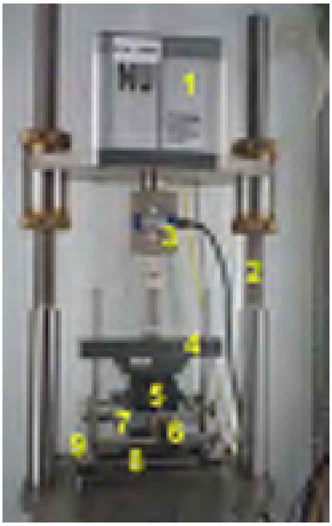
Mixture		AC11surf (1)		SMA 11 (1)		AC 16 surf		SMA 11 (2)		AC11surf (2)		AC16bin		AC22base	
Binder		PmB 45/80-65		PmB 45/80-65		BIT 50/70		PmB 45/80-65		BIT 50/70		BIT 50/70		BIT 50/70	
No. of compaction blows		2*50	2*75	2*50	2*50	2*50	2*50	2*50	2*75	2*50	2*75	2*50	2*75	2*50	2*75
Measurement temp. [°C]	**Measurement direction**	**Number of specimens**													
5; 15; 25; 35	1.	40	0	32	32	32	32	32	32	32	32	32	32	32	32
	2.	40	0	32	32	32	32	32	32	32	32	32	32	32	32

The same principle was applied to testing of four asphalt mixtures type AC 11 conducted in the second part of the research, in which the shares of bitumen and filler in asphalt mixture were varied [4]. Table 2. presents the second part of the asphalt mixtures stiffness modulus testing programme.

Table 2 Stiffness modulus testing programme (second part of testing)

Mixture		AM1	AM2/1	AM2/2	AM3
Binder		BIT 50/70	BIT 50/70	BIT 50/70	BIT 50/70
Number of compaction blows		2*50	2*50	2*50	2*50
Filler Share [%;m/m]		10,7	9,4	9,0	8,2
Bitumen share [%;m/m]		3,6	7,0	5,1	5,1
Share of voids [%;v/v]		8,2	2,0	5,0	5,3
Measurement temperature [°C]	**Measurement direction**	**Number of specimens**			
5; 15; 25; 35	1.	12	12	12	12
	2.	12	12	12	12

All the testing specimens were tested on the device presented in Figure 1. The device and testing equipment are described in detail in the standard HRN EN 12697-26, Appendix C.



1 Pneumatic load impulsator
2 Steel loading frame
3 Load transfer device
4 Top loading bar
5 Test specimen
6 Screws for adjustment of LVDT (linear variable differential transformer)
7 Bearing frame LVDT
8 Bottom loading bar
9 Claws for placement of LVDT frame

Figure 1 Measurement device for asphalt mixture stiffness modulus by application of the indirect tensile test

4 Testing results

Testing results of the first part of testing are shown in Table 3. and Figure 2. The mean values of the stiffness modulus of the tested asphalt mixtures are presented, with differences in compacting of specimens, according to temperatures of measurements. The stiffness modulus obtained on the specimens compacted with 2*75 blows, i.e. on the specimens with less voids, are greater than those of the modulus obtained for the specimens compacted with 2*50 blows. Only the results of AC 16 surf mixture at 5°C, deviate from this rule. Wearing course mixtures have lower stiffness modulus than the mixtures for the binding, base and base-wearing courses. The increase of stiffness modulus following the decrease of temperature is slower in wearing course mixtures.

Table 3 Stiffness modulus values S_m (MPa) of asphalt mixtures, with differences in specimen compaction, in relation to temperatures

Asphalt mixture, Number of compaction blows	Temperature of measurement			
	5°C	15°C	25°C	35°C
SMA 11 45/80-65 (1), 2*75	11656	6047	2640	1052
SMA 11 45/80-65 (1), 2*50	11294	5645	2512	1044
SMA 11 45/80-65 (2), 2*75	11900	5733	2615	1139
SMA 11 45/80-65 (2), 2*50	10536	5217	2150	941
AC 11 surf 45/80-65, 2*50	12945	5535	2838	1133
AC 11 surf 50/70, 2*75	15851	8153	3843	1374
AC 11 surf 50/70, 2*50	14027	7218	3336	1111
AC 16 bin 50/70, 2*75	20029	10307	4499	2082
AC 16 bin 50/70, 2*50	18266	9595	4184	1817
AC 22 base 50/70, 2*75	24519	12595	6517	2446
AC 22 base 50/70, 2*50	21159	10407	4387	1487
AC 16 surf 50/70, 2*75	23327	11684	4556	1601
AC 16 surf 50/70, 2*50	24268	11041	4270	1435

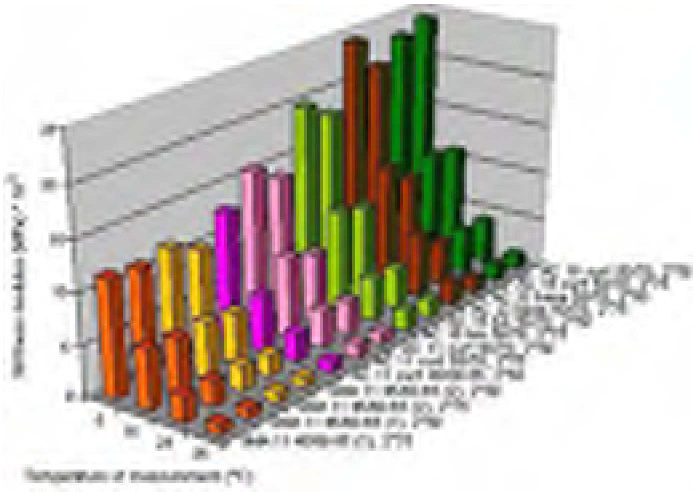


Figure 2 Asphalt mixtures stiffness modulus (MPa), with differences in compacting of specimens, at different temperatures

The stiffness modulus values of all tested mixtures decrease following temperature increase. In case of all tested mixtures the values of modulus obtained at maximal measured temperatures (35°C) are in average 91% lower that the values of modulus determined at the minimal measured temperatures (5°C). The trend of stiffness modulus decrease following temperature increase, in mixtures AC16bin, AC22base and AC16surf, differs from the one in mixtures SMA 11 and AC11surf. Table 4. and Figure 3. present results of the second part of testing.

Table 4 Composition of asphalt mixtures and average stiffness modulus at different test temperatures

Asphalt mixture	Filler share [%;m/m]	Bitumen share [%;m/m]	Share of voids [%;v/v]	Modulus (50°C) [MPa]	Modulus (150°C) [MPa]	Modulus (250°C) [MPa]	Modulus (350°C) [MPa]
AM1	10.7	3.6	8.2	15755	10059	4531	1281
AM2/1	9.4	7.0	2.0	7062	4308	1903	911
AM2/2	9.0	5.1	5.0	11990	7108	2898	857
AM3	8.2	5.1	5.3	11089	7052	2740	1060

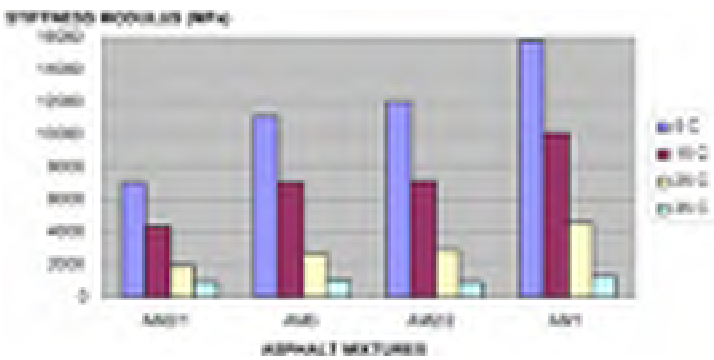


Figure 3 Graphical presentation of asphalt mixtures stiffness modulus at different temperatures

Table 4. shows the mean values of stiffness modulus of individual asphalt mixtures (AM1; AM2/1; AM2/2 and AM3) on test temperatures of 5°C; 15°C; 25°C and 35°C, including respective data on composition of filler and bitumen components and the share of voids in asphalt specimen (same materials in mixtures). The highest values of modulus, on all temperatures, are gained in mixture with the highest filler and voids share and the lowest bitumen share. The lowest values of modulus, on all temperatures, are gained in mixture with the lowest voids share and the highest bitumen share, with relatively high filler share. It can be concluded, from the results of the second part of testing, that values of stiffness modulus generally increase with increase of filler share, and with decrease of bitumen share. The graphical presentation shown below in Figure 3. indicates the trends of stiffness modulus in dependence on composition and properties of asphalt mixture at different temperatures, i.e. behaviour of asphalt at application of load within the indirect tensile impulse testing. The values of stiffness modulus of an optimal asphalt mixture (AM3) are within the medium stiffness area.

5 Conclusion

The stiffness modulus determined at four temperature values confirmed that the indirect tensile testing method enables optimisation of composition and properties of asphalt mixture. Based on the obtained data, the assumptions made during dimensioning shall be closer to the actual behaviour of asphalt pavement structures during exploitation. This will contribute to decreasing of possible deviations from actual values in calculation of stresses and deformities of individual courses due to traffic load and therefore to optimisation of procedure of structural designing of asphalt pavement structures.

References

[1] Dormon, G.M., Edwards, J.M.: “Developments in the application in practice of a fundamental procedure for the design of flexible pavements”, 2nd international conference on the design of asphalt pavements, pp.99–107. 1967.

[2] HRN EN 12697-26, Bituminous mixtures — Test methods for hot mix asphalt — Part 26: Stiffness (EN 12697-26:2004), First Edition, December 2001

[3] Halle, M.: “Temperature impact to asphalt mixture stiffness” Master’s Paper, Faculty of Civil Engineering, Zagreb, 2011 (in Croatian)

[4] Šimun, M.: “New Approach to Waterproofing – Asphalt Pavement System on Bridges” Dissertation, Faculty of Civil Engineering, Zagreb, 2012 (in Croatian)

MOISTURE DAMAGE AND LOW TEMPERATURE CRACKING OF MODIFIED BITUMINOUS MIXTURES FOR ROAD PAVEMENTS

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Abstract

This paper presents an experimental evaluation of moisture damage and low temperature cracking behavior of three bituminous mixtures (Stone Mastic Asphalt, wearing course and base course asphalt concretes), made with steel slag aggregate and different asphalt binders (hard and soft polymer modified bitumen as well as crumb rubber modified bitumen). The asphalt concretes, designed with the traditional Marshall procedure, were tested under aged and unaged conditions. Indirect Tensile Strength Test were used in order to characterize the mixes. The experiments demonstrate that bitumen modified with crumb rubber (from scrap tires), allows to re-use a large amount of steel aggregate (up to 93%, depending on the mixes), with satisfactory results in terms of moisture and low temperature cracking resistance, also in the aged conditions.

Keywords: moisture damage, low temperature cracking, steel slag, crumb rubber, modified bitumen, road pavement

1 Introduction

Moisture damage, as well as thermal cracking at low temperatures, are relevant degradation phenomena that affect the durability and the service life of bituminous mixtures, used for road flexible pavements. The moisture damage resistance of a bituminous mixture is influenced by the content of bitumen and its adhesion to the grains of the lithic skeleton, as well as by the porosity of the asphalt concrete. Binder's rheological properties, especially the ductility, affect the low temperature cracking resistance of the mix.
The paper discusses the results of a laboratory testing concerning the moisture damage and the low temperature resistance of bituminous mixtures (Stone Mastic Asphalt – SMA and traditional ones) for base courses and wearing courses, made with Electric Arc Furnace (EAF) steel slag and three different bituminous binders, modified with fine crumb rubber or SBS polymers. Both aged and unaged samples were tested, in order to investigate the ageing effects.

2 Materials

2.1 Binders

Three different bitumens were used in the experiment for each mixture: Crumb Rubber Modified Bitumen (CRMB), as well as hard and soft SBS polymer modified bitumen (HPMB and SPMB respectively). The fine crumb rubber modified bitumen, as well as the hard and soft polymer modified bitumen, were produced in different private companies' industrial plants.

The asphalt rubber derived from the wet process technology [1]. In this paper, the presence of crumb rubber, hard and soft modified polymers in the mixtures is evidenced by the subscript “ar”, “hm”, “sm” next to the mixture acronym.

2.2 Aggregates

Two granular materials were used in the study: EAF slag and limestone filler. The slags utilized are the main by-product of steel production based on the electric arc furnace (EAF) technology [2]. The EAF slags were supplied by a steel mill in northern Italy, while the limestone filler derived from a quarry in the same area; the steel slags were available in three different fractions: 0/4, 4/8 and 8/14 mm.

2.3 Mixtures

Three mixes were designed: a Stone Mastic Asphalt mix (SMA), a Wearing Course Asphalt concrete (AC W) and a Base course Asphalt Concrete (AC B). The study of the grading curves was referred to the grading envelopes set by SITEB – Italian Society of Bitumen Technologists [3].

3 Methods

3.1 Mix design procedure

The Marshall procedure (EN 12697-34 Standard) was used for determining the optimal binder content. The mixes characterised by maximum bulk density, maximum Marshall Stability, a voids content of 4% for SMA, 5% for AC W, 6% for AC B, were considered optimal. The mechanical performances of the mixtures formulated in this way, were further verified by means of the indirect tensile strength test at 25°C (EN 12697-23 Standard).

3.2 Performance test programme

The performance approach has great relevance in order to properly promote the use of industrial by-products, as outlined also by other Authors [4, 5], therefore the resistance to moisture damage and to thermal cracking have been evaluated, for the asphalt concretes investigated. Indirect tensile strength test at 25°C on wet cylindrical samples have been performed on the nine asphalt concretes optimized in the mix design procedure, in order to investigate the moisture damage resistance. The specimens prepared by Marshall compactor, were furthermore subjected to indirect tensile test at 0°C, in order to evaluate the low temperature cracking. Some of the specimens were exposed to accelerated long-term ageing, by means of conditioning in an oven at 85 °C for 5 days [6], in order to evaluate the effect of seasoning on the mechanical properties of the mixtures and any benefits produced by the bitumen modified with crumb rubber, compared to binders modified with polymers.

4 Results and discussion

4.1 Materials characterization

Table 1 reports the conventional engineering properties of the three bitumens used in the mixes. The elastic recovery data have been certified by the bitumen manufacturer. The crumb rubber modified bitumen, compared to the polymer modified binders, presented improved properties, in terms of penetration (- 24%) and softening point R&B temperature (+ 16%), particularly with respect to the soft modified bitumen. The ductility (evaluated according to the Italian standard) resulted adequate for all the binders.

Table 1 Bitumen characterization

Properties	Standard	CRMB	HPMB	SPMB
Penetration [mm/10]	EN 1426	45	52	59
Softening point [°C]	EN 1427	82	77	71
Ductility [cm]	CNR 44/74	over 100	over 100	over 100
Fraass breaking point [°C]	EN 12593	- 15	- 14	- 12
Elastic Recovery [%]	EN 13398	› 50	› 50	› 50

According to Italian Law, EAF slags are non-hazardous, special non-toxic and non-noxious refuse. They are a solid material, greyish in colour and with no particular smell; the pH is 10.7. By an X-ray fluorescence analysis, it can be observed that the slags contain a prevalence of FeO (33%) and CaO (29%), as well as SiO_2 (13%), and Al_2O_3 (9%). The SiO_2/CaO ratio characterizes the EAF slag as a substantially alkaline aggregate and therefore suitable to guarantee the necessary adhesion with the weakly acid bitumen.
Table 2 reports the physico-mechanical properties of the steel slags, plus the test protocols adopted. A direct test of the volumetric stability of the EAF slags, according to the Standard EN 1744/1 part 15.3, showed an expansion null after the 168 hours suggested in the test protocol. The steel slags demonstrated an excellent affinity with the bitumen, with no stripping of the grains coated with binder after 24 hours of immersion in water at 25 °C (Italian Standard CNR 138/92). This property obviously greatly enhances the durability of the bituminous mixtures.

Table 2 Physical and mechanical characteristics of EAF slags

Properties	Standard	EAF slag 0/4 mm	EAF slag 4/8 mm	EAF slag 8/14 mm
Los Angeles coefficient [%]	EN 1097-2	-	16	14
Equivalent in sand [%]	EN 933-8	86	-	-
Shape Index [%]	EN 933-4	-	1.9	2.9
Flakening Index [%]	EN 933-3	-	4.2	6.4
Freeze/thawing [%]	EN 1367-1	-	0.1	0.1
Fine content [%]	EN 933-1	2.7	0.0	0.0
Grain density [Mg/m^3]	EN 1097-6	3.757	3.719	3.685
Water absorption [%]	EN 1097-6	0.510	0.307	0.112
Stripping in water [%]	CNR 138/92	0	0	0

4.2 Grading and composition of the mixes

Table 3 reports the grading composition of the bituminous mixtures and proportions of the components, while the grading curves of the mixes are presented in Figure 1; the total amount of steel slag was 89%, 92%, 93% for AC B, AC W and SMA respectively. The nominal maximum aggregate size was 15 mm for the SMA and AC B, 10 mm for the AC W.

Table 3 Aggregate type and composition of the mixtures

Mix composition	Fraction [mm]	Quantity [%]		
		SMA	AC W	AC B
EAF steel slag	0/4	45	70	50
	4/8	22	12	13
	8/14	22	10	30
Filler (additive)	–	11	8	7

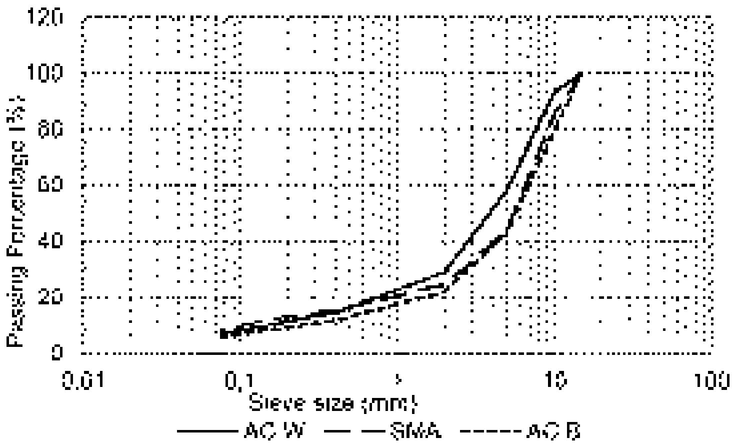

Figure 1 Design grading curves of the mixtures

4.3 Optimization of the mixture

Table 4 reports the Marshall mix design results, along with the Optimum Bitumen Content (OBC) and the Indirect Tensile Strength (ITS) values, of the bituminous mixtures.

Table 4 Mix design results

Mixtures	OBC [%]	VMA [%]	VFB [%]	Bulk density [g/cm³]	MS [daN]	MQ [daN/mm]	ITS [MPa]	ITS (post ageing) [MPa]
SMA/ar	6.0	23.5	82.9	3.306	1,986	482	2.35	2.38
SMA/hm	6.0	23.3	82.8	3.277	1,580	455	2.17	2.23
SMA/sm	6.0	23.1	82.7	3.248	1,324	405	2.04	2.14
AC W/ar	5.0	21.3	76.5	3.338	2,203	523	1.63	1.67
AC W/hm	5.0	21.2	76.4	3.307	1,551	507	1.55	1.62
AC W/sm	5.0	21.0	76.2	3.275	1,530	462	1.46	1.58
AC B/ar	4.0	19.0	68.5	3.325	1,295	450	1.57	1.64
AC B/hm	4.0	18.9	68.3	3.292	1,164	393	1.38	1.48
AC B/sm	4.0	18.7	68.0	3.258	1,222	336	1.23	1.38

For all the mixes the requisites fixed by the Italian Road Technical Standards [3] were guaranteed, with Marshall Stability (MS) and Quotient (MQ) higher than 1,100 daN and 300 daN/mm for AC B, 1,200 daN and 350 daN/mm for AC W, 1,000 daN and 350 daN/mm for SMA. The additional requisite for the SMA, relative to a minimum ITS value equal to 0.6 MPa, was completely satisfied, thus demonstrating the acceptability of these mixtures for road construction. Moreover, for SMA mixes a percentage of Voids in the Mineral Aggregate (VMA) higher than 17% [7] and a percentage of Voids Filled with Bitumen (VFB) between 75% and 85% [1] are needed, whilst for AC W and AC B mixes the requirement is a VMA percentage higher than 15% [8]. According to data in Table 4, all the volumetric requisites are verified for the designed mixes. The mixes with crumb rubber modified bitumen showed better Marshall results as well as ITS values, than the corresponding asphalts with SBS polymer modified binders. All the mixtures presented slightly higher ITS values after the ageing conditioning (up to 12% more than the unaged mix, depending on the asphalt concrete type), even if the hardening phenomenon affects significantly the SBS mixtures more than the crumb rubber asphalts.

4.4 Resistance to moisture damage

The damage caused by the immersion in water is expressed by the Tensile Strength Ratio (TSR). It has been computed as the ratio between the indirect tensile strength of the specimens treated by means of 15 days of immersion in a thermostatic bath at 25 °C (ITS_{wet}) and untreated (ITS_{dry}), respectively. Table 5 presents the ITS_{wet} and the TSR values, for the mixtures studied.

Table 5 Moisture damage – Indirect tensile strength and TSR values

Mixtures	ITSwet [MPa]	ITSwet (post ageing) [MPa]	TSR [%]	TSR (post ageing) [%]
SMA/ar	2.24	2.32	95	97
SMA/hm	1.90	2.06	88	92
SMA/sm	1.73	1.94	85	91
AC W/ar	1.46	1.50	89	90
AC W/hm	1.28	1.36	83	84
AC W/sm	1.17	1.29	80	82
AC B/ar	1.36	1.43	86	87
AC B/hm	1.07	1.18	78	80
AC B/sm	0.94	1.07	76	78

The mixtures made with crumb rubber modified bitumen were clearly characterized by the less pronounced moisture susceptibility, as it is demonstrated by the highest TSR values computed, also in conditions of post-ageing. However it should be stressed that all the asphalts have shown a reasonable resistance to moisture damage. In fact, the limit value of the TSR below which a bituminous mixture may present problems of stripping, usually equal to 70% [1], has been overcome by all the asphalts.

4.5 Resistance to thermal cracking

Materials damage can be expressed by strain energy density function, according to the following relation [9]:

$$\frac{dW}{dV} = \int_0^{\varepsilon_0} \sigma_{ij} d\varepsilon_{ij} \tag{1}$$

where:

σ_{ij} stress;
ε_{ij} strain.

In the indirect tensile test, the area under the stress–strain curve, computed for the peak stress and the corrispondent peak strain, represents the critical value of dW/dV. The critical value of dW/dV at 0°C can be used to estimate the resistance to low temperature cracking [9]. The low temperature cracking test results, namely the peak strain and the critical value of dW/dV, determined for the indirect tensile tests performed at 0°C on the mixtures studied, are presented in Table 6.

The crumb rubber asphalt concretes were characterized by higher peak strain and critical values of dW/dV, with respect to the correspondent polymer modified mixes (increments of dW/dV of up to 117%, depending on the mixture type), thus demonstrating an improved ductility at low temperature and therefore a greater resistance to thermal cracking. The beneficial effect of the crumb rubber modified bitumen is relevant also for the mixtures in conditions of post-ageing.

Table 6 Low temperature cracking test results

Mixtures	Peak strain [%]	Critical value of dW/dV [kJ/m³]	Peak strain (post ageing) [%]	Critical value of dW/dV (post ageing) [kJ/m³]
SMA/ar	1.76	94.74	1.52	72.72
SMA/hm	1.65	72.53	1.12	35.87
SMA/sm	1.42	43.66	0.99	34.77
AC W/ar	1.26	67.00	1.15	58.97
AC W/hm	1.09	39.13	1.07	30.49
AC W/sm	0.93	35.07	0.91	29.32
AC B/ar	1.21	51.83	1.10	49.70
AC B/hm	1.07	35.48	0.90	28.58
AC B/sm	0.89	28.88	0.67	27.66

5 Conclusions

The experimental investigation, conducted on bituminous concretes of the SMA, AC W, AC B type (all made with EAF steel slags), has demonstrated the clearly better performance of the crumb rubber mixtures compared with those of similar type but made with SBS polymer modified bitumen. The results are relevant for the moisture damage resistance, but they are really significant in terms of low temperature cracking resistance. The worst performance have been always recorded for the soft polymer modified mixtures. The effectiveness of the crumb rubber modified bitumen in the increase of moisture damage and low temperature resistance was relevant also for the mixtures in conditions of post-long term ageing. There is a full agreement in the ranking of the mixes, between the different performance tests used in the investigation, namely moisture damage and low temperature cracking tests.

References

[1] Chiu, C. & Lu, L.: A laboratory study in stone matrix asphalt using ground tire rubber, Construction and Building Materials, 21, pp. 1027–1033, 2007.

[2] Liapis, I. & Likoydis S.: Use of electric arc furnace slag in thin skid-resistant surfacing, Procedia – Social and Behavioral Sciences, 48, pp. 907-918, 2012.

[3] Capitolato d'appalto per pavimentazioni stradali con bitume modificato, SITEB, Roma, 2000 [in Italian]

[4] Correira, A.G., Roque, A.J., Ferreira, S.M.R. & Fortunato, E.: Case study to promote the use of industrial byproducts: the relevance of performance tests, ASTM Special Technical Publication, 1540 STP, pp. 267-293, 2012.

[5] Xie, J., Chen, J., Wu, S., Lin, J,. Wei., W.: Performance characteristics of asphalt mixture with basic oxygen furnace slag, Construction and Building Materials, 38, pp. 796-803, 2013.

[6] Artamendi, I., Allen, B,. Phillips, P.: Influence of temperature and aging on laboratory fatigue performance of asphalt mixtures, Advanced Testing and Characterization of Bituminous Materials conference, Rhodes, pp. 185-194, 2009.

[7] National Asphalt Pavement Association, Guidelines for materials, production, and placement of stone matrix asphalt (SMA), Technical Working Group (TWG), Publication N. IS 118, 1994.

[8] Asphalt Institute, Superpave Level 1 Mix Design, Superpave Series N. 2, USA, 1995.

[9] Wu, S., Xue, Y., Ye, Q. & Chen, Y.: Utilization of steel slag as an aggregate for stone mastic asphalt (SMA) mixtures, Building and Environment, 42, pp. 2580-2585, 2007.

COMPARISON THE CHARACTERISTICS OF AC 8 SURF AND AC 11 SURF AND RESULTS BETWEEN TREE LABORATORIES AT LOW TEMPERATURES

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Abstract

The first part of this paper presents comparison of the characteristics of asphalt concrete AC 8 surf and AC 11 surf, with bitumen B 50/70, at low temperatures. The second part shows the results of international comparison between three laboratories in EU, who carried out the Tensile Stress Restrained Specimen Test (TSRST) and Uniaxial Tensile Strength Test (UTST) in accordance with standard EN 12697-46. In the first part are presented results of the TSRST and UTST tests as a function of the asphalt mixture characteristic (bitumen content and voids filled with bitumen – VFB) and temperature. The analysis of the results has shown that there are certain differences, especially at the higher temperatures of the test. In the second part, it is shown that there are obvious differences in the results at TSRST and UTST test between compared laboratories. Based on these findings, we believe that it would be necessary to do more extensive inter-laboratory analysis to confirm this hypothesis and that there is need to update the standard EN 12697-46 in some points.

Keywords: asphalt concrete, bitumen, low temperatures, cracks

1 Introduction

The asphalt samples expand when they are heated and contract when they are cooled. If the contraction due to cooling is prevented with falling temperatures increasing tensile stresses in the asphalt material will be generated, which can lead to fracture (micro-cracking in the binder matrix) if the maximum tensile strength is reached [1]. These damages are primarily due to temperature changes, changes in bituminous binder, excessive traffic load and/or deficiencies in construction and maintenance. So, the knowledge of the behaviour of cracks is essential both for researchers, planners and civil engineers.

This paper presents comparison of the characteristics of asphalt concrete AC 8 surf and AC 11 surf, with bitumen B 50/70, at low temperatures. It is shows the results of the Tensile Stress Restrained Specimen Tests (TSRST) and Uniaxial Tensile Strength Test (UTST) as a function of the bitumen content in the asphalt mixture and temperature. In the second part shows the results of international comparison between tree laboratories in EU, who carried out the TSRST and UTST in accordance with standard EN 12697-46.

TSRST test simulates the condition of asphalt pavement at low temperatures, where the resulting thermally induced tensile stresses, called cryogenic stress, primarily reflect as transverse cracks spaced at 3 to 5 m (Fig. 1) [2]. UTST test simulates the resistance of asphalt mixtures at low temperatures exposed to traffic loading. The maximum of critical tensile stress does not occur in the wheel track but in a distance of 30 cm to 90 cm from the location of loading (Fig. 2).

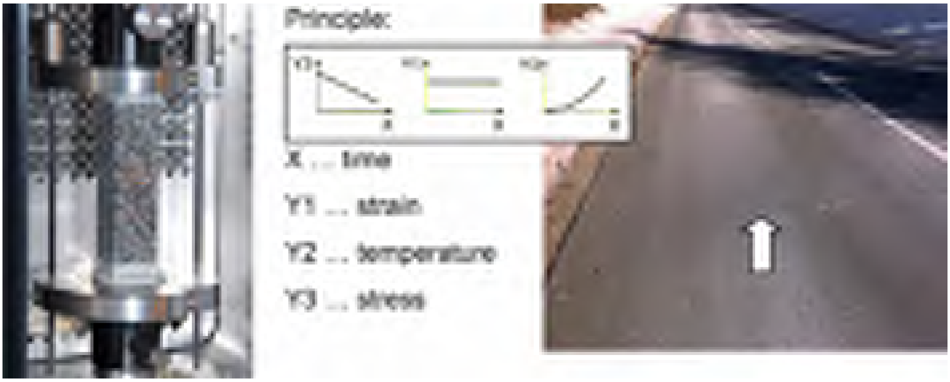

Figure 1 Tensile Stress Restrained specimen Test – TSRST (left) and transversal thermal crack (right) [3]

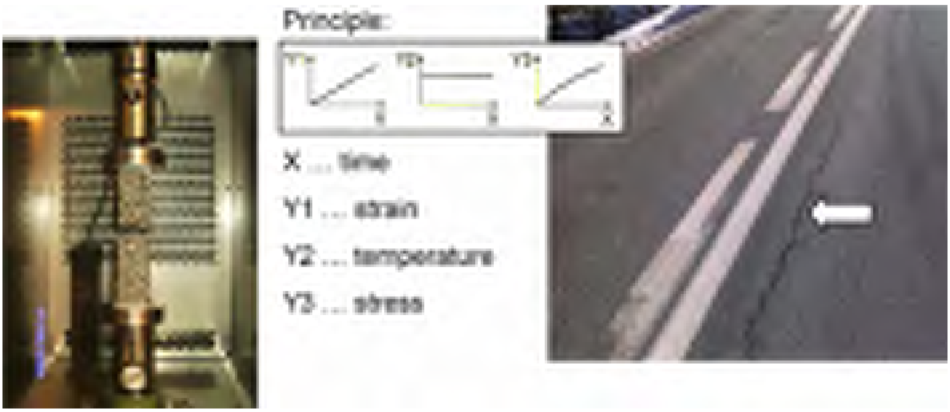

Figure 2 Uniaxial Tensile Strength Test – UTST (left) and longitudinal crack at the wheel tracks (right) [3]

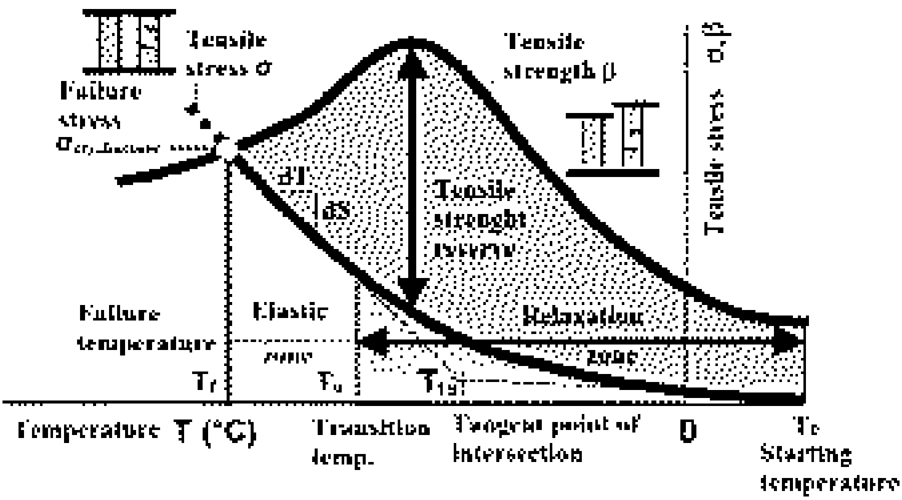

Figure 3 The Arand (1987) concept of tensile stress with relaxation and elastic zone [2, 3, 6]

The [4] concept of tensile stress, tensile strength and tensile strength reserve is shown in Fig. 3. The thermally inducted (cryogenic) stress in asphalt specimen gradually increases as temperature decreases, until the specimen fractures [5]. At the break point, the stress reaches its maximum value – the fracture stress $\sigma_{cry,fracture}$ at fracture temperature $T_{failure}$ (hereinafter T_f). At lower temperatures the slope of the stress-temperature curve dS/dT becomes constant, and the curve is linear (elastic behaviour). The transition temperature T_u divides the curve into two parts – relaxation zone and elastic zone (non-relaxation) and tangent point of intersection T_{TS} is intersection between tangent of the stress-temperature curve at elastic zone and relaxation zone. The bitumen in asphalt specimen becomes stiffer when the temperature approaches the transition temperature and the thermally inducted stresses are not relaxed below this temperature [6]. The difference between the tensile strength and low temperature stress is known as the tensile strength reserve $\Delta\beta_t(T)$ and it is the reserve that is available to accommodate additional superimposed stresses (traffic inducted stress) [5]. Equation for tensile strength reserve is:

$$\Delta\beta_t(T) = \beta_t(T) - \sigma_{cry}(T) \tag{1}$$

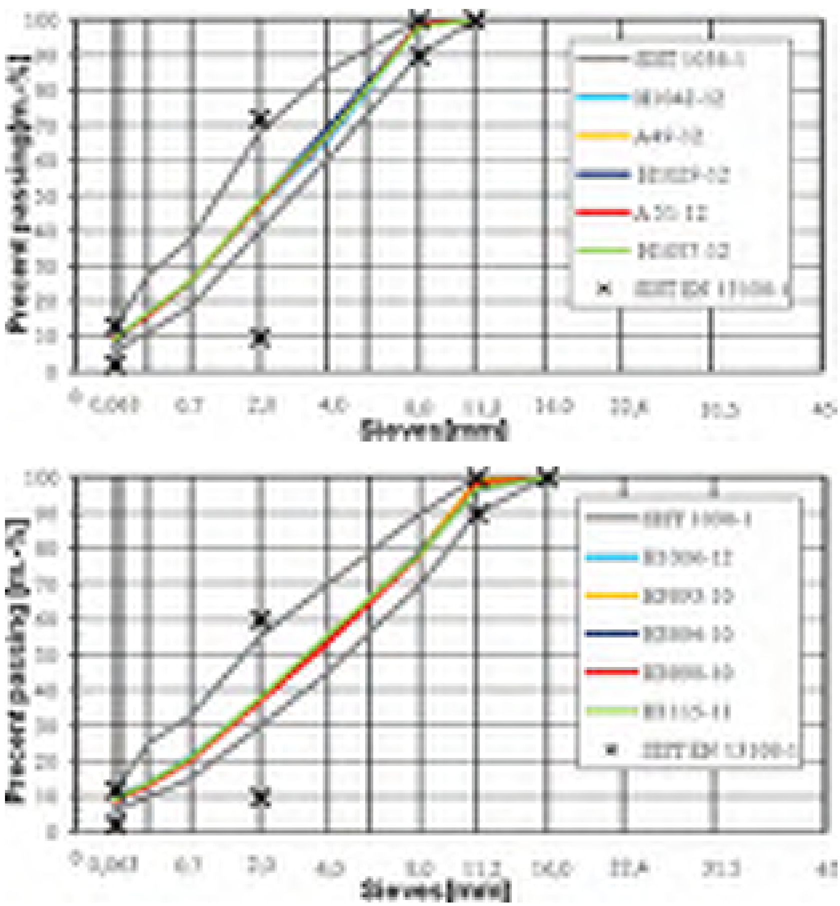

Figure 4 Grain size distribution of AC 8 surf and AC 11 surf

2 Comparison AC 8 surf and AC 11 surf at low temperatures

2.1 Material

The tests were carried out on asphalt concrete 0/8 mm (AC 8 surf) and asphalt concrete 0/11 mm (AC 11 surf). The asphalt mixture and test specimens were prepared in the laboratory ZAG Ljubljana. In the laboratory we prepared five different mixtures with bitumen content of 4.0, 5.0, 5.4, 5.8 and 6.0 (at AC 11 surf), 6.2 (at AC 8 surf) m.-%. Stone fractions used for the stone aggregate mixture are as follows: filler aggregate (grain under 0.125 mm) from Stahovica (limestone), mineral aggregate 0/2, 2/4 and 4/8 mm from Ljubešćica (silicate). For binder we used paving grade bitumen 50/70 by MOL (Hungary). Fig. 4 presents grain size distribution for this asphalt mixture AC 8 surf and AC 11 surf. The grain size distributions overlap and it is clear that the filler aggregate was practically constant. For tests we used a rectangular specimen with cross section dimensions 40 x 40 mm^2 and a length of 160 mm by the EN 12697-46. Table 1 shows properties of used fresh and extracted paving grade bitumen 50/70. Binder was extracted from asphalt with trichloroethylene Infratest Asphalt Analyzer (EN 12697-1). In our research, we decided to try to keep a constant content of filler aggregate in asphalt mixture and to vary air voids. Table 2 presents the results of some basic tests of asphalt mixtures AC 8 surf. Table 3 presents the results of some basic tests of asphalt mixtures AC 11 surf.

Table 1 Properties of paving grade bitumen 50/70

Technical characteristics	Unit	Test method	Fresh bitumen	Extracted bitumen
Penetration at 25ºC	mm/10	EN 1426:2007	58	37
Softening point, R&B	°C	EN 1427:2007	50	55.6
Penetration Index, PI	–	EN 12591:2004,B4	-0.4	-0.57
Fraass braking point	°C	EN 12593:2007	-8	-7
Density (in water)	kg/m^3	EN ISO 3838	1020	–
BBR – stiffness S60	°C	EN 14771:2005	-15.6	–
BBR – m-value60	°C	EN 14771:2005	-17.8	–

Table 2 Results of some basic tests of asphalt mixtures AC 8 surf

Bitumen content [m.- %]	Grain size ‹0.063 mm [m.-%]	Bulk density of Marshall spec. [kg/m3]	Air voids of Marshall spec. [V.-%]	Voids filled with bit. VFB [V.-%]
4.0	9.1	2411	8.4	53.0
4.9	9.0	2420	6.5	56.7
5.4	8.6	2457	4.6	73.4
5.8	9.1	2442	3.0	76.1
6.2	8.7	2483	2.3	86.7

Table 3 Results of some basic tests of asphalt mixtures AC 11 surf

Bitumen content [m.- %]	Grain size ‹0.063 mm [m.-%]	Bulk density of Marshall spec. [kg/m3]	Air voids of Marshall spec. [V.-%]	Voids filled with bit. VFB [V.-%]
3.9	9.3	2404	8.0	62.8
4.9	9.1	2441	5.0	79.4
5.3	9.0	2461	3.8	86.1
5.6	8.6	2467	2.6	92.5
6.0	9.0	2484	1.8	97.9

2.2 Results and analysis

TSRST and UTST test results depending on the content of bitumen in asphalt samples are shown in this section. Intentionally other influences on the results (e.g. equipment, preparation ...) were excluded. In Fig. 5 are shown the results of the test TSRST. The samples of asphalt mixture AC 8 surf have higher tensile stress at failure $\sigma_{cry,f}$ than samples of asphalt mixture AC 11 surf. From almost constant tensile stress it can be concluded that the content of bitumen in the asphalt mixture does not significantly effect on tensile stress. As it was expected temperature tensile stress T_f results for asphalt mixture AC 8 surf is lower than for AC 11 surf, due to higher content of grains under 2 mm.

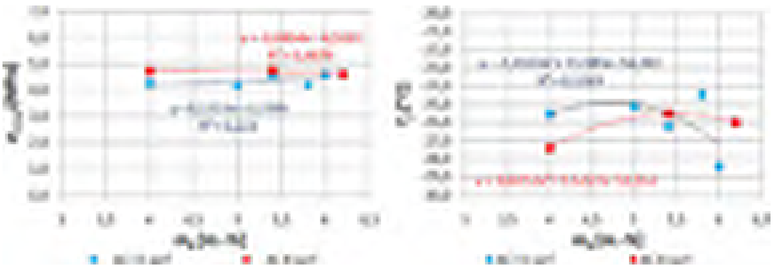

Figure 5 Tensile stress of failure $\sigma_{cry,f}$ (left) and failure temperature T_f (right) results depending of the bitumen content (TSRST test)

Differences between asphalt mixture AC 11 surf and AC 8 surf at UTST test are small when tensile strength β_t is measured (Fig. 6). At test temperature +20 °C the tensile strength β_t of both asphalt mixtures is similar. At lower test temperatures the tensile strength of AC 8 surf samples is becoming higher compared with AC 11 surf. It is assumed that larger grains of aggregate take more strength at higher temperature. From almost constant tensile strength it can be concluded that the increasing of bitumen content in the asphalt mixture does not significantly effect on tensile strength in the zone of optimal bitumen content.

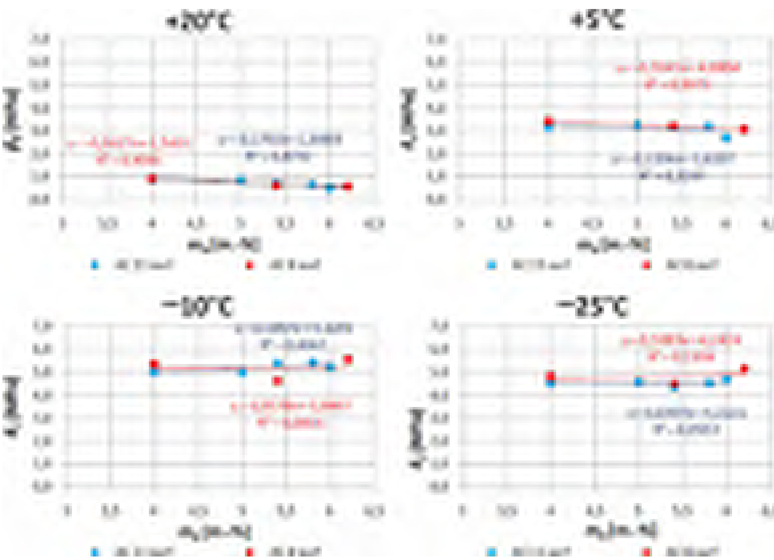

Figure 6 Tensile strength β_t results depending of the bitumen content m_b (UTST test)

There is some dependency between failure strain ε_t – and content of bitumen m_b at higher temperatures (Fig. 7). At test temperature +20 °C and +5 °C we see that the content of bitumen has influence on strain. Also the differences between AC 8 surf and AC 11 surf are obvious at these temperatures. The asphalt mixture AC 8 surf has higher strain ε_t then AC 11 surf. At test temperature –10 °C and –25 °C the stiffness of both mixtures is a similar and elastic. Failure strain is more related to the properties of material than strength, but in the standard only tensile strength β_t is required.

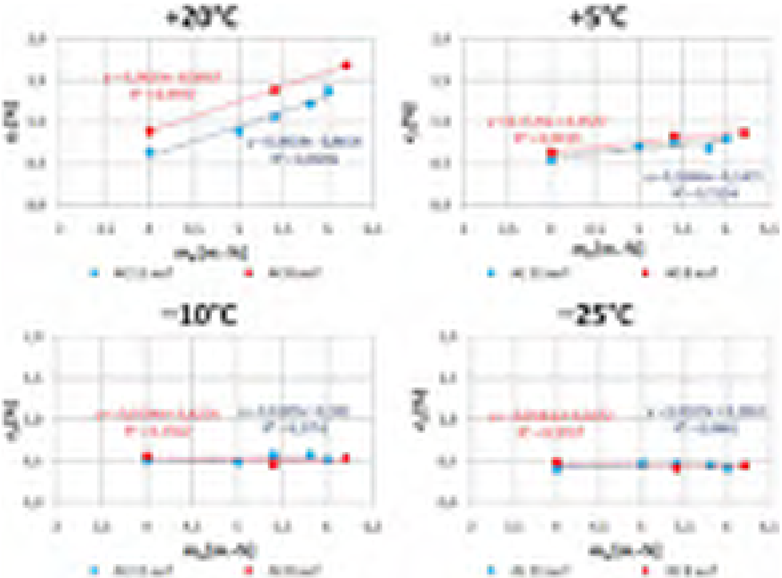

Figure 7 Failure strain depending of the bitumen content between AC 8 surf and AC 11 surf at UTST test

Fig. 8 shows the results of maximum tensile strength reserve $\Delta\beta_{tmax}$ and temperature at maximum tensile strength $T_{\Delta\beta tmax}$. Only temperature at maximum tensile strength $T_{\Delta\beta tmax}$ is significantly different for both asphalt mixtures. The results for asphalt mixture AC 11 surf are better, because of higher values of VFB in the mixture.

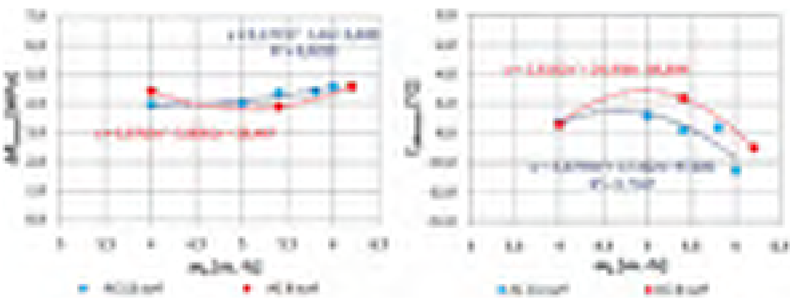

Figure 8 Maximum tensile strength reserve $\Delta\beta_{tmax}$ (left) and temperature at maximum tensile strength $T_{\Delta\beta tmax}$ (right) results depending of the bitumen content m_b

3 Comparison between tree laboratory at low temperatures

In this section is presented inter-laboratory comparison between three independent laboratories in EU. Laboratory 1 is TU WIEN from Austria, Laboratory 2 is ZAG Ljubljana from Slovenia and third Laboratory 3 is RAMTECH from Croatia. All three laboratories carried out low temperature tests according to standard EN 12697-46. All AC 8 asphalt mixtures and test specimens were prepared in Slovene laboratory. The asphalt samples tested in Laboratory 1 had 5.0 m.-%, 5.8 m.-% and 6.2 m.-% of bitumen, asphalt samples tested in Laboratory 2 had 4.0 m.-%, 5.4 m.-% and 6.2 m.-% of bitumen and asphalt samples tested in Laboratory 3 5 m.-% and 5.8 m.-% of bitumen.

In Fig. 9 are presented diagrams of TSRST tests carried out in all three laboratories. The results from Laboratory 1 are presented with red curves, results from Laboratory 2 with blue curves and from Laboratory 3 with green curves. From Fig. 9 two basic shapes of curves can be seen. The first type of curves are coming from Laboratory 2 with 4 m.-%, 5.4 m.-%, 6.2 m.-% and Laboratory3 with 5.0 m.-% of bitumen content. The second type of curves are coming from Laboratory 1 and one curve from Laboratory 2 with 5.8 m.-%. Common to second type of curves is that they all have temperature of failure T_f under – 31 °C, on the contrary first type of curves doesn't reach less then – 27 °C. So, the results are out reproducibility limits required by standard EN 12697-46 (ΔT_f = 2 °C). The tension stress (at all curves with equal content of bitumen) are inside reproducibility limits required by standard ($\Delta\sigma_{cry}$ = 0.5 MPa). Relaxation zone at second type of curves is longer, representing better behaviour of material at low temperatures.

Results of UTST tests are shown in Fig. 10. The highest maximum of tensile strength β_t and the steepest slope of curves were measured in Laboratory 1 (red curves). On the contrary in Laboratory 3 (green curves) the lowest maximums of tensile strength β_t were measured. The differences between results from these two laboratories are significant due to the fact that they both used the same type of test equipment.

The curves of tension strength reserve for all laboratories are presented in Fig. 11. As it was expected from results of UTST tests the highest maximum of tensile strength reserve $\Delta\beta_{tmax}$ were obtained in Laboratory 1 (red curves) and the lowest in Laboratory 3 (green curves).

In Laboratory 1 were obtained significantly better results for low temperature resistance than in other two laboratories. All samples were prepared in the same way, so influence of the sample preparation on the results was minimized. The only difference we noticed was at gluing samples. All this three laboratories has different procedure of gluing samples. In standard EN 12697-46 procedure of gluing samples is not described precisely enough.

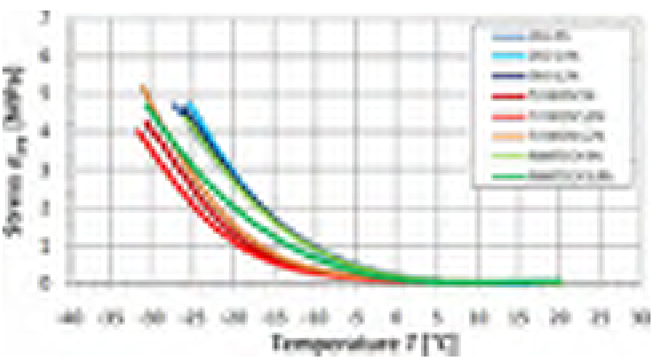

Figure 9 Diagram of a TSRST (tensile stress – temperature) test for AC 8 surf

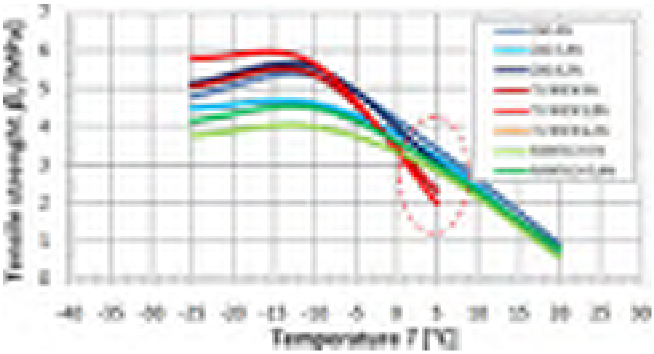

Figure 10 Diagram of a UTST (tensile strength – temperature) for AC 8 surf

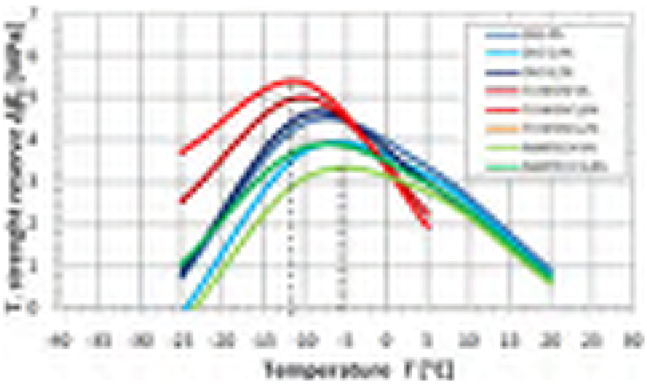

Figure 11 Diagram of a tensile strength reserve – temperature for AC 8 surf

4 Conclusion

The first part of this paper is presented comparison of the characteristics of asphalt concrete AC 8 surf and AC 11 surf, with bitumen B 50/70, at low temperatures. The results of the Tensile Stress Restrained Specimen Tests (TSRST) and Uniaxial Tensile Strength Test (UTST) as a function of the bitumen content in the asphalt mixture and temperature were studied.
With this study mostly insignificant differences between low temperature properties of these two asphalt mixtures were found. Only at test temperature +20 °C (UTST) significant differences between AC 8 surf and AC 11 surf were found. Variation in the bitumen content in the mixture has minor influence on tensile stress or tensile strength in the range of the optimal bitumen content. At test temperature +20 °C and +5 °C it was found that the content of bitumen has influence on strain. Failure strain is more related to the properties of material than strength, but in the standard EN 12697-46 only tensile strength β_t is required.
In this section is presented inter-laboratory comparison between three independent laboratories in EU, where TSRST and UTST tests were carried out. We found out that two laboratories have the same equipment, but different results were obtained. All samples were prepared in the same way, so influence of the sample preparation on the results was minimized. The only difference we noticed was at gluing samples. All this three laboratories has different glue and procedure of gluing samples. At this point standard EN 12697-46 is not precise enough. In the standard it would be necessary to more accurately describe the glue, who have constant properties (coefficient of expansion) in the range of test temperature.

References

[1] The Shell Bitumen Handbook, 5th edition, Tomas Telford Publishing, London, pp. 196-199, 2003.

[2] Spiegl, M.: Tieftemperaturverhalten von bituminösen Baustoffen – Labortechnische Ansprache und numerische Simulation des Gebrauchsverhaltens. Dissertation, Institut für Straßenbau und Straßenerhaltung, Technische Universität Wien, Heft 19,Wien, pp. 13-17, 2008.

[3] Hribar, D., Tušar, M.: Properties of asphalt concrete at low temperatures, Journal GRADEVINAR 64 (2012) 10, pp. 825-831.

[4] Arand, W.: Kälteverhalten von Asphalt; Teil 1: Bewertungshintergrund zur Beurteilung des Verhaltens von Walzasphalten bei Kälte; Teil 2: Einfluss der Zusammensetzung auf das Verhalten von Walzasphalten bei Kälte. Die Asphaltstraße – Das stationäre Mischwerk 21, 1987.

[5] SIST EN 12697-46:2012 Bituminous mixtures – Test methods for hot mix asphalt – Part 46: Low Temperature Cracking and Properties by Uniaxial Tension Tests

[6] Kanerva, H. K., Vinson, T.S., Zeng, H.: SHRP – Strategic Highway Research Program: Low-Temperature Cracking: Field Validation of the Thermal Stress Restrained Specimen Test. (SHRP-A-401), National Research Council, Washington, pp. 7-8, 1994. http://onlinepubs.trb.org/onlinepubs/shrp/SHRP-A-401.pdf (January, 2013)

CETRA 2014 28–30 April 2014, Split, Croatia
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EXAMPLES OF REUSE OF MATERIALS OF DECONSTRUCTION FOR THE CONSTITUTION OF A ROAD STRUCTURE – RECYVIA® PROCESS

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Eurovia, France

Abstract

Maintenance of the existing road network is a critical point for every stake older. The main issues are very often: a) economical solution in order to aloud a good service level on the whole network with structural decrease of dotation budgets and b) environmental friendly solution in order to preserve the materials and the environment. With the background of these 2 main issues, recycling solutions are excellent solutions : economical aspect (it is cheaper to re-use instead of milling/evacuation/renew operation) and environmental aspects (less transport, less energy consumption, less fumes emission for cold solutions, smaller global carbon footprint ...). Of course, deep analyses have to be undertaken on the road condition (structural and surface conditions) as well as on the materials themselves (relevant analyses in Laboratory). Road conditions measurements will permit a good pavement design corresponding to the traffic and lifetime data for the reinforcement or maintenance works.
On the second hand, the analyses on the materials of the existing layers will define the possible ways of recycling. Indeed, the type of recycling (cement, bituminous emulsion, bituminous foam, association of hydraulic and bituminous binders...) and the re-use conditions will be defined from the results of the Laboratory analyses (grading, fine content, binder content, ageing, residual mechanical performances...). The aim of the paper is to explain how the best solution is found (definition of the technic and design of the whole reinforcement or maintenance structure) from the measurements done on the existing road.

Keywords: recycling, economy, environment, sustainability, technical solution

1 Introduction

Eurovia has developed since the 90's a range of processes that can be used in the capping layer or road base layer course which responds to the problems of the renewal of the roads of several clients under the form of simple, efficient and economical processes.
The RECYVIA® [1] range unites products and processes that consist in in-situ treatment (up to 32cm thick by pass of treatment) or a plant of aggregates, road asphalt pavement coming from planing or crushing, crushed concrete from civil engineer bridges. These materials are treated with a hydraulic binder, an emulsion or a foam bitumen. This mixture has the sufficient mechanical performances in order to ensure a veritable structural contribution and to present an excellent behavior under high traffic. The thickness of the surface layer is to be adapted to the solicitations of the road.
The most of the recycled materials can be inserted in the Recyvia® composition, under reserve of a previous study in laboratory and that one reaches a rate of 50 to 100% of recycled materials in the mixture. The paper presents the genesis of this product, its technical and environmental characteristics and describes some real laying examples.

2 Overview

Within the Eurovia Group, the cold retreatment has been developed by the Canadian subsidiary DJL since the 90ies with the RECYFLEX®: recycled coarse gravel based on crushed concrete, road asphalt pavement coming from planing or crushing and sometimes new aggregates. The materials are treated cold in a plant with the help of emulsion of bitumen combined with a hydraulic binder. The processes of cold road retreatments are part of this evolution. They allow:

- the natural resources preservation: up to 100% of recycled materials;
- the reduction of polluted emissions: use of cold processes reduces importantly the emission of CO2. For a warm mix one needs 10 to 14 liters of fuel per ton, less than one liter is necessary for the production of a ton of asphalt concrete with emulsion;
- the improvement of the environment of the jobsite: reduction of the inconvenience for the users thanks to a short delay of the jobsite, reduction of fumes and reduction of heavy traffic linked to the jobsite.

Another important key benefit of these techniques is, of course, their lower costs. The reuse of materials as well as the cold production allows effectively a considerable saving compared to conventional processes of road refurbishment.
Like every process, the cold treatment presents certain limits. Especially, they are not adapted in case of a presence of an important number of buried networks, it is necessary to carry out studies and test to qualify the support, which can increase the costs of the works.

3 Technical characteristics

In 2009, Eurovia united its processes of cold retreatment for road base layer courses under the range of RECYVIA®. With the integrated binder, the materials have sufficient mechanical performances in order to ensure a veritable structural contribution in order to guarantee the requested sustainability of the roads:

- one can privilege an emulsion or a foam bitumen for the roads the structural capacity of which is high. The presence of a hydraulic binder is aimed at obtaining good characteristics at a young age of the retreated material during the phase of maturation of the cold bituminous treatment;
- the unique contribution of the hydraulic binder allows an important structural reinforcement of the road and improves the mechanical properties of the material.

Recyvia® is mainly used for refurbishment of existing roads; this is its proper definition. However one can plan in the case of new roads by using recycled materials coming from other road or civil engineering jobsites. A complete mix design study, taking into account the characteristics of the used materials, is carried out. This study concerns especially the following points:

- complete characterization of the materials to be recycled (grading, cleanliness, density, penetration and residual ring and ball temperature of the binder);
- index on characteristics Proctor on the mixture;
- tests of mix design relative to the mixture in order to determine the dosing of the binder;
- mechanical characteristics of the mixture.

The previous study of the Recyvia® jobsite is compulsory in order to adapt the mix design to materials of the former structure. Note that the most of the recycled materials from the road can be inserted in the composition of Recyvia®. Binder and additives are adjusted in quality and quantity in order to reach the calculated performances.The Recyvia® in-situ process calls for a specific workshop of retreatment consisting of the most performing available equipment on the market, as the final setting is ensured with a grader (see Fig. 1). The laying of Recyvia® in a plant is carried out in a tradional way with the grader or paver according to its composition (see Fig. 2 and 3). In both cases, the compaction is ensured by a design workshop regarding the materials to be compacted and its thickness.

Figure 1 In-situ Treatment train Recyvia®, Route 202 in LACOLLE (Quebec)

Figure 2 Plant for production of Recyvia® in the bypass of Troyes (France)

Figure 3 Laying of Recyvia® in the bypass of Troyes (France)

4 Some jobsite examples

4.1 Bypass of Troyes

The jobsite of the bypass North of Troye (10 – France) was carried out in summer / autumn 2010 [2]. The road pavement structure was:
- Surface course: 2,5 cm VTAC / Binder course: 10 cm SCAC / geotextile / 20 cm RECYVIA® with pre-cracking Olivia / 19 cm RECYVIA®.

Note: regarding the structural contribution, in this case, Recyvia® is nearly equivalent to a cement bound aggregate class T3 [3]. The road base course and the base layer are carried out with Recyvia® process characterized for this jobsite by a high percentage of recycled coarse gravel. A laboratory study allowed to determine the formula as follows:
- Contribution of fillerized sand 10 %
- Recycled coarse gravel 86,5 %
- Hydraulic binder 3,5 %

Samples were made in order to obtain the mechanical characteristics of the materials at 60 and 360 days. The tensile resistance [4] R_t and the elasticity modulus [5] E are measured. An estimation of the performances at 360 days was carried out according to the ratios:
- $R_{t60}/R_{t360} = 0.65$;
- $E_{60}/E_{360} = 0\ .70$, (EN 14227-5 page 4).

Table 1 provides the obtained values at 60 days and those estimated at 360 days.

Table 1 Measuring results on RECYVIA for the bypass North of Troye jobsite

	3.5% ROC SC	
	R_t [MPa]	**E_t [MPa]**
60 days	0.995	25572
360 days (estimation)	1.658	36531
Class after 360 days (est.)	T3	

The process of anti-reflective cracking is implemented for the inherent transversal cracking on hydraulic bound materials. It consists of a pre-cracking according to the Olivia® process and the geogrid Rotaflex 816 SL waterproofed with a bitumen emulsion dosed at 1.5 kg/m². The binder course, with semi-coarse asphaltic concrete, is produced "warm" according to the Evotherm® process and with the incorporation of 15% reclaimed asphalt pavement.

4.2 Other jobsites

A number of jobsites was carried out within the Eurovia group; while declining locally the Recyvia®. One can note for example:

- Vancouver (Canada): Recyvia® in-situ: 32,000 m^2
  - Works on 2x2 lanes of an urban boulevard, with heavy traffic and bus stops, emergency lanes and road widening in crossroads.
  - Retreatment of the foam bitumen (+ 1% of cement Portland) on a thickness of 20cm

- Martin County – Florida (USA): Recyvia® in-situ: 116,000 m^2:
  - Reconstruction of the CR 609 SW Allapattah Road, on a length of 17 km. Jobsite carried out within 60 days (see Fig. 4).

Figure 4 Martin County jobsite in Florida : before, during and after

5 Environmental interests

A base solution of the road structure can be compared to an alternative solution equivalent using Recyvia® in order to better take up the environmental contribution. The proposed example is the one of the Troyes bypass, in this case, Recyvia® is a treatment in a hydraulic binder plant that contains 86.5% of recycled materials. Its use, combined with the incorporation of RAP and the use of the Evotherm® MA3 process in order to produce semi-coarse asphaltic concrete that builds the binder course allows to obtain an environmental report more than positive compared to the base solution. Some points are particularly remarkable.

5.1 Transport

The retreatment of road materials in general and more particularly the in-situ retreatment, allows to reduce importantly the transport for the jobsite, as this transport represents more than 25% of the consumed energy and the production of greenhouse gas for a traditional road jobsite. Note that in the case of a retreatment in a treatment plant, the production tool, compact and mobile, is easily adapted at every type of jobsites: this flexibility allows its installation next to the works, reducing so the transport. In our example, this reduction is 35% and reaches 81% regarding the local road transport.

5.2 Energy savings

The cold retreatment allows important savings on the fossil energy consumption (fuel or gas) because of the absence of reheating of the aggregates components. 22% of the direct fuel consumption and 17% of the energetic resource consumption are saved.

5.3 Emission of greenhouse gas

In absence of the heating of the recycled aggregates components, the Recyvia® process does not emit in the atmosphere the greenhouse gas coming from the residual bitumen of the reclaimed asphalt pavement and allows in our example to reduce by 19%.

6 Conclusions

Studies are currently in progress in order to better know the behavior of Recyvia® depending on the type of the used binder, on the incorporated dosings and on type of the base materials. They will allow then to optimize the Recyvia® characteristics and to carry out a cartography of possible users of this process in the different entities of Eurovia all over the world.
Since the beginnings of this process in the 90ies in Canada, this process was applied by certain companies of the Eurovia group regarding the regulations and technical habits of every country. The feedbacks of the jobsites already carried out show that the process can respond to every type of traffic, from communal roads to highways with very high traffic while reducing the user's inconvenience (limits of the jobsite traffic by using the in-situ materials …).
Taking into account the environmental preoccupation more and more strong, the wish to reduce the greenhouse gas and to limit the consumption of non-renewable materials, this process should be generalized by the clients within the scope of heavy refurbishment works (road refurbishment of the slow lane with high traffic, taxiways) but also the creation of new roads by using the materials of deconstruction of civil engineering (crushed concrete, sleepers …). The use of the eco comparators in the judgment of the offers should also allow to highlight the undeniable environmental characteristics of the Recyvia® process.

References

[1] Recyvia is a trade mark belonging to Eurovia

[2] Article in “Route Actualités” 07/08/2010

[3] According to standard NF EN 14227-1

[4] According to standard NFEN 13286-42

[5] According to standard NFEN 13286-43

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ENVIRONMENT PROTECTION BY USING NEW TECHNOLOGIES FOR ASPHALT MIXTURES

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Abstract

The asphalt paving industry is constantly exploring technological improvements that will enhance the material's performance, increase construction efficiency, conserve resources, and protect the environment. Current and impending regulations on greenhouse gas emissions, fumes/odors and energy conservation are making attractive the reductions in asphalt mix production and placement temperatures. Warm mix asphalt (WMA) refers to asphalt concrete mixtures that are produced at temperatures approximately equal to 100°C – 140°C bellow temperatures typically used in the production of hot mix asphalt (HMA). The goal of WMA is to produce mixtures with similar strength, durability, and performance characteristics as HMA using substantially reduced production temperatures. This paper aims to presents a laboratory study performed on a few Romanian asphalt mixtures manufactured in both technologies (hot and warm). The performances of these asphalt mixtures (WMA) are studied by their characteristics like Marshall flow and stability, stiffness and permanent deformation. All these tests are performed according to European Norms. The results are presented like comparative graphs and the conclusions will establish which is the performance of WMA.

Keywords: warm mix asphalt, CO_2 reduction, bitumen additives, mixture stiffness

1 Introduction

The asphalt paving industry is constantly exploring technological improvements that will enhance the material's performance, increase construction efficiency, conserve resources, and protect the environment. Current and impending regulations on greenhouse gas emissions, fumes/odors and energy conservation are making attractive the reductions in asphalt mix production and placement temperatures.
In these conditions, the concept of warm mix asphalt (WMA) has a good opportunity. Warm mix asphalt is produced at temperatures in the range of 20 to 40ºC lower than typical hot mix asphalt (HMA). WMA represents a group of technologies that allow a reduction in the temperatures at which asphalt mixes are produced and placed. These technologies tend to reduce the viscosity of the bitumen and provide complete aggregate coating at lower temperatures. The same mechanisms that allow WMA to improve workability at lower temperatures also allow WMA technologies to act as compaction aids. Improved compaction or in-place density tends to reduce permeability and bitumen hardening due to aging, which tends to improve performance in terms of cracking resistance and moisture susceptibility. WMA technologies also have the potential to be beneficial during cold-weather paving or when mixtures must be transported long distances before placement. The smaller differential between the mix temperature and ambient temperature results in a slower rate of cooling. Since WMA can be compacted at lower temperatures, more time is available for compaction. Classification of asphalt mixtures based on manufacturing temperature is presented in table 1.

Figure 1 Typical mixing temperature for asphalt mixtures

Table 1 Manufacturing temperatures

Technology	Mixing temperature [°C]	Technology principle	Influence on bitumen
"HOT MIX"	120 – 190	Hot bitumen	
"WARM MIX" ("WMA")	100 – 140	Organic additives	Bitumen and mixture rheology are modified
"COLD MIX"	No heating	Bitumen emulsion	Reduce bitumen viscosity

Development of this technology appear in Europe, in 1997, at the same moment with the Kyoto agreement negotiations, which purpose was to reduce pollutant emissions by industrialized countries by 5,2% between 2008 – 2012, comparatively with those from 1990. Since then on the construction market appears a lot of products and technology processes which promise to implement the directives from Kyoto Agreement, products and technologies that are currently the subject of several laboratory studies internationally.
Also, it is possible by reducing temperatures mixing and placing, using additives which lower bitumen viscosity, to improve the compaction characteristics of asphalt mixture layer, by assuring a better density. This will lead to a permeability reduction of asphalt mixture layers, to a better aging behavior with favorable effects on fatigue resistance and moisture degradation of structure layers.

1.1 Review of the Existing Warm Mix Asphalt Systems

1.1.1 Foaming techniques

In the presence of hot bitumen, a small quantity of water (~ 2% by mass of bitumen) changes from liquid to vapors. The rapid expansion of the water, from liquid to vapors, creates thin-film bitumen bubbles filled with water vapors referred to as foamed bitumen. In a foam state, the viscosity of the bitumen is reduced allowing full aggregate coating at lower mixing temperatures. The characteristics of foamed bitumen are described in terms of expansion ratio (volume of foam compared to volume of liquid) and the half-life of the foam which provides a description of the stability of the foam in time.

1.1.2 Bitumen emulsion

The bitumen emulsion technique was developed in North America and it consists of mixing a specific high residue bitumen emulsion with hot aggregate at a reduced mixing temperature. As the emulsion is mixes with the hot aggregate the water flashes off as steam. The bitumen emulsion is specifically designed for the WMA process and includes additives to improve coating, workability and adhesion. It has been reported that mixture workability remains excellent at relatively low temperatures (‹ 80°C) [1],[2], which is a specific benefit of this WMA system.

1.1.3 Organic additives

Organic additives in the form of wax have emerged successfully from an extensive program of laboratory and field trials as a bitumen modifier that enables mixing and compaction at reduced temperatures. Organic additives are often referred to as "intelligent fillers" as they provide reduced viscosity at mixing/placement temperatures and increased viscosity at service temperatures, which is an added benefit specific to this type of WMA system. Furthermore, these additives increase the viscosity of the binder, thus providing increase in resistance to deformation at high ambient temperatures. These type of additives act as an compaction aid if the amount is small – less the 1,5% and they can improve bitumen properties is the amount is higher the 3%.

1.1.4 Chemical additive

Warm mix asphalt systems involving the use of chemical additives or surfactants are not relying on the reduction of the binder viscosity, but rather the improvement of the coating capability of the binder at a lower mixing temperature. The WMA systems using this approach are relatively new and their development is promising. Certain chemicals are added to the binder in manner similar to anti-stripping agents in a concentration as low as 0.3 % by mass of the bitumen. These chemicals can be pre-blended in bitumen or added at plant pumping installation.

2 Laboratory studies

Experimental study aimed that through laboratory results to highlight asphalt mixture performance when WMA technology is used. To evaluate mechanical performance of asphalt mixtures, the cylinder specimens (100.6 mm diameter and 63.5 mm high) were produced with 50 blows compacting energy per side by Marshall Compactor, and the slabs were prepared on wheel-roller machine. The size of the slab samples was 300 mm in length, 300 mm in width and 50 mm in thickness. This study was carried out on two types of asphalt mixtures for wearing course designed according to Romanian Norms: an asphalt mixture – noted by "HMA" and an asphalt mixture produced with Rediset organic additive (WMA type) – noted by "WMA". The materials (aggregates, fibber and bitumen) used to prepare the asphalt mixtures and the asphalt mixtures recipes are the following:

- crushed rock from Cerna:
- 8/16: 30% by mix
- 4/8: 22% by mix
- 0/4: 32% by mix
- filler from Holcim: 9.4 % by mix;
- bitumen 50/70 plus: 6.6% by mix;
- organic additive (Rediset): 2% by bitumen – used for WMA mix.

The organic additive used is a wax (brown flakes) added in bitumen, dissolving easy when bitumen is hot, without any special mixing equipment. To highlight the performance of "warm mix" mixture type, in Roads Laboratory of Faculty of Railways, Roads and Bridges (Technical University of Civil Engineering of Bucharest) were conducted these tests:

- Marshall test on cylindrical specimens, according to SR EN 12697-34, at 60°C test temperature for Marshall stability and index flow;
- Test applying Indirect Tension to cylindrical specimens IT-CY, according to SR EN 12697-26 Annex C and SR EN 12697-24 Annex E, at 20°C test temperatures from which result asphalt mixture stiffness;
- Cyclic Compression test on cylindrical specimens, according to SR EN 12697-25, for triaxial compression, at 50°C test temperature from which result resistance to permanent deformation;
- Wheel Tracking test on slabs, according to SR EN 12697-22, small device, method B in air at 60°C test temperature from which result resistance to permanent deformation.

The European standards for bituminous mixtures (EN 13108-1 to -7) do not preclude the use of WMA. The standards include maximum temperatures for particular mixtures, but there are no minimum requirements. The minimum temperature of asphalt mix is declared by the manufacturer. The standards also allow usage of additives if the performance of asphalt is equivalent to reference mixture. In tables 2 and 3 are presented temperatures used at preparing and compaction of asphalt mixture and bitumen properties. In figures 2, 3 and tables 4, 5, 6, 7 are presented the results obtained trough experimental program.

Table 2 Temperatures used at preparing and compacting of asphalt mixtures

Temperature, °C	Technology	
	"hot mix"	"warm mix"
Aggregates	180	150
Bitumen	170-175	170-175
Mixing	180	160
Compaction	170-175	125&150

Table 3 Bitumen characteristics

Characteristic	Bitumen type	
	50/70 plus	50/70 plus with wmx additives
Penetration, at 25°C, 1/10 mm	51	48
R&B point, °C	52	56
Penetration index	-1.0	-0.5

Table 4 Physical – mechanical characteristics

Characteristic	Asphalt mixture type		
	HMA	WMA1	WMA2
	Compaction temperatures		
	160 .. 165 °C	125 °C	150 °C
Density, kg/m^3	2327	2326	2374
Water absorption, %	5.5	5.52	4.23
Marshall Stability (S), kN	9.85	7.69	12.15
Marshall Flow (l), mm	3.6	2.9	3.0
Marshall Index (S/l), kN/mm	2.74	2.65	4.05
Stiffness, IT-CY, 20°C, MPa	3416	2803	3530

In the future it is a requirement to consider the impact that our actions will have on the environment, especially with regards to the preservation and sustainability of the environment for the future generations. The most obvious environmental advantage with regards to the use of warm mix asphalt for road construction is decreasing the temperature in the production of WMA, which will lower fuel usage and decrease emissions directly connected to fuel use (table 8). This should lower the emissions of greenhouse gases (CO_2) and traditional gaseous pollutants (CO, NO_x, and SO_2).

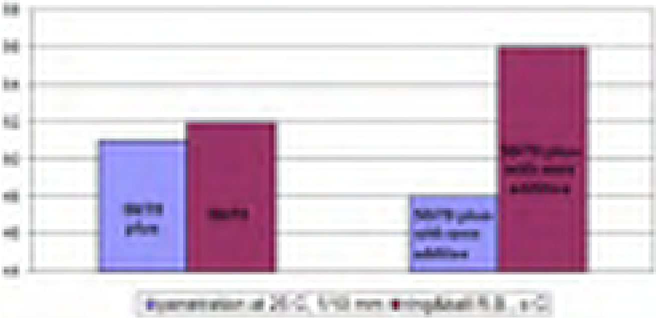

Figure 2 Working technology influence on bitumen properties

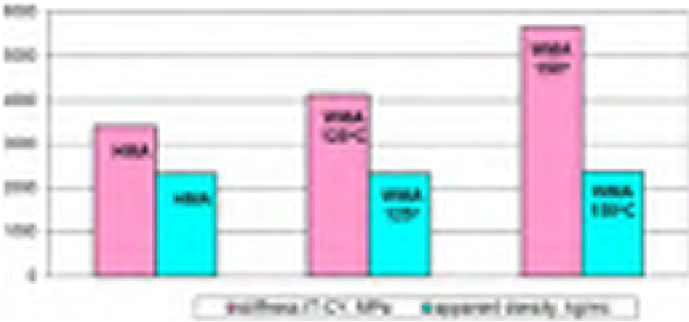

Figure 3 Working technology influence on asphalt mixture properties

Table 5 Resistance to permanent deformation (method I)

Type of mixture	Parameters of equation on (quasi) linear stage II Method I ($\varepsilon_n=A_1+B_1 n$)		Creep rate $f_c=B_1$	Initial creep modulus, $E_n=\sigma/\varepsilon_n$, kPa
	A_1	B_1		
HMA	5129.1	0.0478	0.0478	1243
WMA	11420	0.039	0.039	407

Table 6 Resistance to permanent deformation (method II)

Type of mixture	Parameters of equation on (quasi) linear stage II Method II (logεn=logA+Blogn)		Calculated permanent deformation	
	A	B	ε_{1000}	ε_{10000}
HMA	3096.71	0.0463	4828	5599
WMA	9440	0.0208	10899	11433

Table 7 Resistance to permanent deformation from Wheel Tracking test

Characteristics	Type of mixture	
	HMA	**WMA**
Proportional rut depth at 10^4 load cycle (%)	2.56	2.34
Wheel Tracking slope (mm/10^3 load cycle)	0.029	0.025

Table 8 Calculated reduction of CO2 emission at the one of the mixture plant in Romania

Indicators	HMA	WMA
Fuel consumption (l)	1.426	1226
CO_2 emission (t)	3.342	2.685
Mixture quantity (t)	155	140
CO_2 emission per ton of mixture (kg CO_2/t)	21,56	19.18 (-11.04%)

According to the measurements, this gives a plant stack emission reduction of:
- CO_2 in the range of 15% to 40%;
- SO_2 – 20% to 35%;
- volatile organic compounds (VOC) up to 50%;
- carbon monoxide (CO) – 10% to 30%;
- nitrous oxides (NOX) – 60% to 70%.

3 Conclusions

Warm mix asphalt is a promising approach to production and placement of paving materials. Research work worldwide is evidently demonstrating that WMA systems are providing significant benefits with regards to the environment, in facilitating paving practices and, with regards to field performance. Significant evaluation work has been completed and the benefits associated with WMA are well documented. However, there is still a significant challenge ahead to move WMA from trial projects to main stream pavement products. All WMA systems are either proprietary processes or based on specific commercial products. Specifications that allow fair competition between the various WMA systems remain to be developed. Using the "warm mix" technology, the working temperatures can be reduced by keeping the initial physic – mechanical characteristics of the asphalt mixtures obtained through warm technology "hot mix".
Warm mix type mixture has a poorer behavior at permanent deformation resulted from traxial compression test because of the additive which decrease bitumen viscosity, but still fits in demand of norm. For example, ε_{10000} values is double in case of "warm mix". In return, adding of additive in asphalt mixture improve the resistance to permanent deformation from Wheel Tracking test by about 10%.
Increasing the compaction temperature, it can be noticed an improvement of the physic – mechanical characteristics obtained through the technology "warm mix", and also the results have satisfactory values. Still, the best results are obtained for a compaction temperature of 150°C. Close values of the physical – mechanical characteristics of the mixtures achieved through "hot mix" technology and "warm mix" technology are obtained for the compaction temperature of 125°C.
Although the laboratory results appear to indicate some small changes in the performance of the mixtures, they do not consider aging effects and the performance functions are based on conventional mixture data, and therefore need to be validated by field data before any conclusions can be drawn.

References

[1] D'Angelo, J., Harm, E., Bartoszek, J., Baumgardner, G., Corrigan, M., Cowsert, J., Harman, T., Jamsihidi, M., Jones, W., Newcomb, D., Prowell, B., Sines, R., Yeaton, B.: Warm-Mix Asphalt: European Practice. American Trade Initiatives. Washington, DC: U.S. Department of Transportation, 2008.

[2] Prowell, B.D., Hurley, G.C.: Warm-Mix Asphalt: Best Practices, QIS 125, National Asphalt Pavement Association, Lanham, MD, USA, 2008.

[3] Răcănel, C., Burlacu, A., Moon, K.H., Mărăşteanu, M.: "Investigation of warm mix asphalt field and laboratory prepared mixture", ISAP 2012, 2nd International Symposion on Asphalt Pavements & Environment, Fortaleza, Brazilia 2012.

EFFECTS OF A CHEMICAL WMA ADDITIVE ON AGING CHARACTERISTICS OF BITUMINOUS MIXTURES

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Abstract

Utilization of warm mix asphalt (WMA) in pavement construction is a relatively new technology, therefore there is nearly no chance to practically evaluate the long-term characteristics of the existing WMA pavements. Due to lower application temperatures, it can intuitively be expected that a WMA pavement would be less subjected to aging-induced failures. This study attempts to evaluate the effects of a chemical WMA additive on aging characteristics of bituminous mixtures. The chemıcal additive tested within the scope of this study is a combination of both organic additive and a kind of cationic surfactant developed by Akzo Nobel. Short- and long-term aging conditions were simulated on mixtures containing various contents of chemical WMA additive as well as on control specimens. To estimate the proportions of hardening, aging indices were determined based on indirect tensile strength (ITS) results. The aging indices unveiled that the specimens containing chemical WMA additive demonstrate higher resistance against hardening than conventional hot mix asphalt (HMA) specimens.

Keywords: warm mix asphalt; chemical WMA additive, short term aging; long term aging; indirect tensile strength

1 Introduction

Most of the field pavement practices around the world consist of the conventional Hot Mix Asphalt (HMA). For last decade, implementing of WMA technologies has gained popularity in Europe and some other countries as well as in the United States of America. Nowadays, many new WMA technologies have been introduced to the market. One of the well-known chemical WMA additives in the market is Rediset® WMX developed by Akzo Nobel which is a combination of both organic additive and a kind of cationic surfactant. This study attempts to evaluate the effects of this chemical WMA additive on aging characteristics of bituminous mixtures.
Premature failures of asphalt pavements are often caused by diminishing of the binding forces in the mixture by the effect of moisture or mechanical stresses or aging of bitumen. The durability of asphalt concrete is a measure of its level of resistance to hardening (also called aging) over time [1]. The effect of time and environmental factors on the aging of bitumen is a subject of continuous interest [2]. Earlier studies conducted on the effect of temperature and relative humidity on the oxidation of bitumen as a function of exposure time showed that the rate of oxidation is dependent on both of these environmental factors [2], [3]. The effect of ambient temperature on the ageing characteristics of the bitumen is rather well documented. Petersen (1984) pointed out that three basic factors control the changes that would result in ageing in bitumen [4]. First factor better called volatilization is the loss of the oily components of bitumen by volatility or by absorption of porous aggregates. Second factor called oxidation is known as the Change in the chemical composition of bitumen resulted from reaction with

oxygen. Third factor known as steric hardening is molecular structuring that produces thixotropic effects. Among these factors, oxidation is mainly considered as a major factor contributing in the hardening of the asphalt pavement.
The literature review shows that aging of asphalt occurs in two terms called Short- and Long-term in order [5]. Short-term ageing occurs during mixing and final placement when the asphalt is at elevated temperatures. This is probably caused by volatilization. Long-term aging occurs partially throughout the short-term aging frame and while the asphalt is in service and exposed to environment. This long-term aging is predominantly caused by oxidation. Aging of asphalt may be influenced simultaneously by several factors, such as characteristics and content of the bitumen, nature and particle size distribution of the aggregates, void content of the mixture, production related factors, and external conditions such as temperature and time [6]. Asphalt additives would considerably change the mixture characteristics. Beside the effect of modification in bitumen, it can intuitively be expected that a WMA pavement would be less subjected to aging than a HMA pavement in construction and hauling phases.

2 Experimental

2.1 Materials

The bitumen used in this study was 50/70 penetration grade bitumen which was provided from TUPRAŞ Aliağa refinery. This grade of penetration is commonly used in İzmir; Turkey due to climatic conditions. Conventional test results for virgin bitumen conducted at laboratory of bituminous materials in Dokuz Eylül University are given in Table 1.
A mix of basalt and limestone aggregates provided from Dere Madencilik Inc. (Quarry located in Belkahve – İzmir) is used in this study. Physical properties of each kind are given in Table 2. After conducting the associated tests based on ASTM C 136, a mix gradation of basalt and limestone is intentionally chosen to provide desired performance in conformity with Turkish specifications concerning the Type 1 wearing course. Basalt plays the role of strengthening constituent as coarse aggregate while limestone participates in fine aggregate framework. The gradation is given in Table 3.

Table 1 Laboratory test results for virgin bitumen

Test	Standard	Results	Turkish Specifications
Penetration (25°C ; 0.1 mm)	ASTM D5	55	50-70
Softening Point (°C)	ASTM D36	49	46-54
Viscosity (135°C)	ASTM D4402	412.5	–
Viscosity (165°C)	ASTM D4403	137.5	–
TFOT (165°C)	ASTM D1754		
Mass change (%)		0.04	‹0.5
Penetration Change (%)	ASTM D5	25	–
Softening Point after TFOT (°C)	ASTM D36	54	48‹
Ductility (25°C; cm)	ASTM D113	100	–
Specific Gravity	ASTM D70	1.03	–
Flash Point (°C)	ASTM D92	+260	230‹

The chemical WMA additive tested in this study is Rediset® WMX which is a combination of both organic additive and a kind of cationic surfactant [7]. The surfactants simply increase the coating ability of the aggregate with the bitumen by "active adhesion." and the other constituents have role in reducing the viscosity of the bitumen [8], [9]. Using this additive can reduce the production temperature by about 10°C to 15°C and consequently results in 20%

reduction in fuel consumption [8], [10]. This additive can either be added to the bitumen or to the mixture. Plant modification is not needed or minor changes are sufficient [8]. The supplier recommendation about the sufficient dosage is 1.5% to 2% by the virgin bitumen weight [11].

Table 2 Physical properties of aggregates

Test	Specification	Limestone	Basalt	Limits
Specific gravity (coarse agg.)	ASTM C 127			
Bulk		2.686	2.666	-
Saturated Surface Dry		2.701	2.810	-
Apparent		2.727	2.706	-
Specific gravity (fine agg.)	ASTM C 128			
Bulk		2.687	2.652	-
Saturated Surface Dry		2.703	2.770	-
Apparent		2.732	2.688	-
Specific gravity (Filler)		2.725	2.731	-
Los Angeles Abrasion (%)	ASTM C 131	24.4	14.2	‹ 45
Flat and elongated particles (%)	ASTM D 4791	7.5	5.5	‹ 10
Sodium sulfate soundness (%)	ASTM C 88	1.47	2.6	10-20
Fine aggregate angularity	ASTM C 1252	47.85	58.1	40‹

Table 3 Mixed gradation of basalt and limestone

Test	19 – 12.5 mm (Basalt)	12.5 – 5 mm (Basalt)	5 – 0 mm (Limestone)	Combined (%)	Limits
Mix ratio (%)	15	45	40		
Gradation					
(3/4)”	100	100	100	100	100
(1/2)”	35.7	100	100	90.5	83-100
(3/8)”	2.5	89	100	80.5	70-90
No 4	0.4	16	100	47.3	40-55
No 10	0.3	1.2	81	33	25-38
No 40	0.2	0.7	33	13.5	10-20
No 80	0.15	0.4	22	9	6-15
No 200	0.10	0.2	13	5.3	4-10

2.2 Experimental Plan

Based on the regarding production temperature and mixing times for chemical WMA additive, WMA bitumens were produced just before the mixing with aggregates process. Production temperature was supplied by a heater similar to Thermosel™ and controlled by a digital industrial thermometer. An industrial stirrer was used in production of WMA bitumens. Stirring was done in normal shear stresses (1000 rpm by a stainless steel stirrer bar with about a 4 cm cross crown) since the production of WMA bitumens don’t demand for high shear stresses. Mixing temperatures and periods were determined based on trial and error method. 15 minutes production period at 150 °C temperature was determined as optimum process.
Mixing and compaction temperatures for chemical WMA additive modified bitumen were derived from equiviscous method in accordance to AASHTO T 312. The graphics for viscosities vs. temperature were plotted for all additive contents. Acceptable temperature for mixing was chosen as the range matching 0.17±0.02 Pa·s and acceptable temperature for compaction was chosen as the range matching 0.28±0.03 Pa·s. For various additive contents of chemical

additive by virgin bitumen weight, mixing and compaction temperatures are given in Table 4. Data for virgin bitumen used in HMA is presented by 0 % (no additive) in the table.

Table 4 Mixing and compaction temperatures

Additive Content (%)	Mixing Temp. (°C)	Compaction Temp. (°C)
0	157 – 164	144 – 150
1	151 – 157	138 – 144
2	145 – 149	136 – 140
3	144 – 147	133 – 138

Following the production of chemical WMA additive modified bitumens, WMA mixtures were prepared based on the determined mixing temperatures. The industrial mixer used for mixing the aggregates and the bitumen. The optimum bitumen content was determined by Marshall mix design method. The optimum bitumen content for both virgin bitumen and chemical WMA additive modified bitumen were respectively determined as 4.76% and 4.46% by weight of aggregates. The aggregates were placed in an oven adjusted for proper temperature the day before to be completely dried and ready for mixing. Specimens were compacted with Marshall compactor regarding their compaction temperatures after mixing process. Conditioning as per AASHTO R 30 standard were done on the specimens intended to be aged. Short-term aged specimens were conditioned in a forced-draft oven set for 135°C for 4 hours and then compacted and cured since the long-term aged specimens were conditioned for 124 hours in a forced-draft oven set for 85°C after passing the short-term aging conditions. All processes regarding the conditioning of the mixtures are shown on a flowchart in Fig. 1.

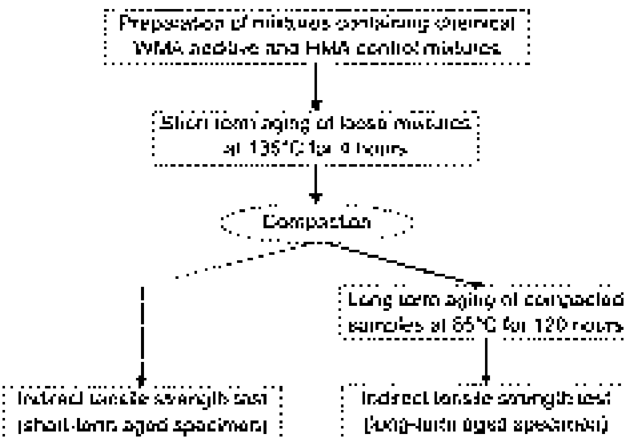

Figure 1 Short- and long-term conditioning flowchart

All specimens were tested for Indirect Tensile Strength (ITS) after being cured. To perform this task, ASTM D6931 -the standard test method for indirect tensile strength of bituminous mixtures- has been taken into account. The ITS test was conducted by Marshall stability and flow apparatus. The loading rate was set to 51(mm/min) in case for ITS. To be adequate and unbiased, three specimens for all additive contents were prepared and tested randomly. The ITS results can give the evaluation keys in terms of low temperature and fatigue cracking of asphalt pavements. Some studies introduce ITS result as a good indicator in predicting laboratory rutting potential of asphalt mixtures [12].

The bitumen becomes more brittle and stiffer as it ages, thus the ITS results of an aged mixture are rather more than the results of an un-aged mixture. This fact can simply provide us with aging indices to investigate the aging characteristics of asphalt mixtures. Burak Sengoz (2003) has implemented ITS results of mixtures with various air voids, to assess aging and moisture susceptibility characteristics of HMA mixtures [13]. Another study on short- and long-term aging behavior of rubber modified asphalts has also proved the fact that the short-term and long-term aging increased the measured tensile strengths [14]. Sengoz and Topal (2008) investigated the effects of SBS polymer modified bitumen on the aging properties of asphalt mixtures using ITS results [15]. They calculated aging indices as the ratio of short- and long-term aged specimen's ITS values to the values of un-aged control specimens prepared with the same additive content.

3 Results and discussions

The ITS results for un-aged, short- and long-term aged specimens are given in Figure 3. In this figure, additive content corresponding 0 simply demonstrates the HMA control specimens. The results of un-aged specimens indicate that using Rediset® WMX can potentially increase the indirect tensile strength of a specimen. This fact is observable in additive contents more than 2%. Higher ITS value in case of un-aged specimens points out the specimen strength in terms of internal bonds. This fact is in conformity with previous studies that all stated about the "active adhesion." which plays role in increasing the coating ability of the aggregate with the bitumen by means of surfactants and consequently can result in higher internal bonds [8], [16].

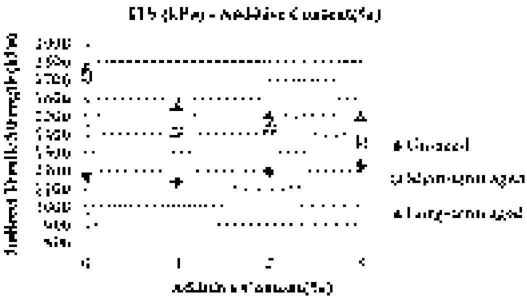

Figure 2 ITS results corresponding additive contents

As seen in the Figure 3, the differences between the ITS values of un-aged and aged specimens obviously decrease by increasing in additive content. This fact is basically born of aging effects of use of chemical WMA additive. Aging indices can provide better evaluation for the effects of chemical WMA additive on aging characteristics of bituminous mixtures. In this study, these indices were calculated as the ratio of ITS values of short-term and long-term aged specimens over ITS values of HMA control specimens. The calculating formulas for Short-term Aging Index (SAI) and Long-term Aging Index (LAI) are as follows:

$$\mathrm{SAI} = \frac{\text{ITS value of short} - \text{term aged specimen}}{\text{ITS value of un} - \text{aged specimen}} \tag{1}$$

$$\mathrm{LAI} = \frac{\text{ITS value of long} - \text{term aged specimen}}{\text{ITS value of un} - \text{aged specimen}} \tag{2}$$

By means of these indices, we are able to observe how hardened an aged specimen has become after the aging process. The less an aging index of a mixture is, the more resistive that mixture is against hardening. The SAI and LAI values for all contents of chemical WMA additive are demonstrated in Figure 4.

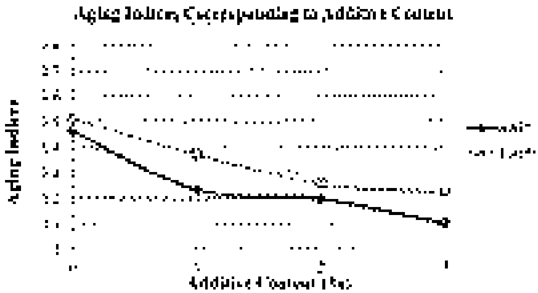

Figure 3 ITS results corresponding additive contents. Note: * 'SAI': Short-term Aging Index; ** 'LAI': Long-term Aging Index

As seen in the graph, chemical WMA additive modified mixtures exhibited lower aging indices than control specimens as it was expected. As discussed before, temperature is a vital factor in occasion of the aging. Lower application temperatures can cause less aging especially in case of short-term aging. Having a glance on the slope of the trendlines for both SAI and LAI values, It can simply be realized that the drop in SAI values is more than the drop in LAI values. This can be explained considering the effect of application temperatures on aging of bituminous mixtures during construction phase. The short-term aging occurs during production, storage and hauling in particular temperatures. These temperatures differ for WMA and HMA pavements, since the long term conditions for both pavement types are mostly equal during the service life of the pavements.
Chemical WMA additive modified mixtures demonstrated better strength against aging in comparison with HMA mixtures. Test results showed that less hardening occurred when the additive content increased. The reduction slope seemed to become less in step by step increasing in additive content.

4 Conclusion

Asphalt pavement has a service life. The more the service life of an asphalt pavement, the more economic that asphalt pavement is. From economical point of view, simply considering the initial cost of a structure is not efficient and it is also vital to consider the long term characteristics of the structure in service. Although WMA technologies have a lot of economic benefits in comparison to HMA pavements in terms of energy, it is essential to consider the long term properties of WMA pavements. As aforementioned, this study was allocated to evaluate the effects of a chemical WMA additive on WMA pavements in terms of long term and short term aging.
As a conclusion, generally WMA technologies can be categorized as acceptable alternatives for conventional HMA pavements in terms of aging. Utilization of WMA additives in lower temperatures is the dominating advantage of WMA technologies from the aging point of view. Rather than lower application temperatures, based on catalogue information, chemical WMA additive contains anti-stripping agent within its chemical structure which can play role in reducing aging effects parallel to reducing stripping.

References

[1] Bell, C.A.: Summary Report on Ageing of Asphalt Aggregate Systems, Oregon State University, Corvallis, 1989.

[2] Campbell, P.G., Wrigth J.R. & Bowman P.B.: The Effect of Temperature and Humidity on the Oxidation of Air Blown Asphalts, Materials Research Standards, 2, pp. 988-995, 1962.

[3] Chipperfield, E. H.: Asphalt Characteristics in Relation to Road Performance, Annual Journal of Association of Asphalt Paving Technologists, 39, pp. 79-88, 1970.

[4] Petersen, J.C.: Chemical Composition of Asphalt as Related to Asphalt Durability: State of Art, Transportation Research Record, 999, pp. 13-30, 1984.

[5] Bell, C.A., Wieder, A.J. & Felin, M.J.: Laboratory Ageing of Asphalt Aggregate Mixtures: Field Validation – SHRP-A-390, Oregon State University, Corvallis, 1994.

[6] Xiaohu, L. & Isaccson, U.: Artificial ageing of polymer modified bitumens, Journal of Applied Polymer Science, 76, pp. 1811-1824, 1999.

[7] Chowdhury, A. & Button, J. W.: A Review of Warm Mix Asphalt, Texas A&M University System College Station, Texas Transportation Institute, Texas, 2008.

[8] Lai, J. S. & Tsai, J. Y.: Evaluating Constructability and Properties of Advera and REVIX Warm Mix Asphalt: Final Report, Georgia Dept. of Education Office of Materials & Research, Forest Park, Ga, 2008.

[9] Prowell, B. D. & Hurley, G. C.: Warm Mix Asphalt: Best Practices, National Asphalt Pavement Association, 2007.

[10] D'Angelo, J., Harm, E., Bartosze, J., Baumgardner, G., Corrigan, M., Cowsert, J., Harman, T., Jamshidi, M., Jones, W., Newcomb, D., Prowell, B., Sines, R. & Yeaton, B.: Warm Mix Asphalt: European Practice, FHWA Report No. FHWA-PL-08-007, American Trade Initiatives, Alexandria, Virginia, 2008.

[11] Rubio, M. C., Martinez, G., Baena, L. & Moreno, F.: Warm Mix Asphalt: an Overview Journal of Cleaner Production, 24, pp. 76-84, 2012.

[12] Anderson, R. M., Christensen, D. W., & Bonaquist, R.: Estimating the Rutting Potential of Asphalt Mixtures Using Superpave Gyratory Compaction Properties and Indirect Tensile Strength, (With Discussion), Journal of the Association of Asphalt Paving Technologists, 72, pp. 1-26, 2003.

[13] Sengoz, B.: The Effect of Asphalt Film Thickness on the Aging and Moisture Susceptibility of Hot Mix Asphalt, PhD Thesis, Istanbul Technical University, Graduate School of Natural and Applied Sciences, Istanbul, 2003.

[14] Liang, R. Y. & Lee, S.: Short-Term and Long-Term Aging Behavior of Rubber Modified Asphalt Paving Mixture, Transportation Research Record, 1530, pp. 11-17, 1996.

[15] Topal, A. & Sengoz, B.: Effect of SBS Polymer Modified Bitumen on the Ageing Properties of Asphalt, Proceedings of the 4th Eurasphalt and Eurobitume Congress, Copenhagen, pp. 1147-1153, 2008.

[16] Hurley, G. C. & Prowell, B. D.: Evaluation of Sasobit® for Use in Warm Mix Asphalt, US: National Center for Asphalt Technology, Auburn, 2005.

IMPACT OF SELECTED CHEMICAL ADDITIVES ON PERFORMANCE BEHAVIOR OF WARM ASPHALT CONCRETE MIX

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Abstract

Warm mix asphalts are becoming a regular group of mixtures used in pavement structures in many European countries. Nevertheless there are still a lot of mind barriers which disallow a more distinctive use of this type of mixes especially with respect to improved healthy conditions for road workers and because of potential energy savings during mixing of such asphalt mixture. In parallel there are still ambiguities about possible weaknesses with respect to durability or cracking resistance. Last but not least the actual market offers quite large variety of different additives which needs suitable comparison and proof of real benefits of particular additives. Within the experimental study done as part of ongoing research at the Czech Technical University in Prague different chemical additives have been used representing synthetic waxes or surfactants, as well as comparing them with ready-to-use binders offered by some producers. Besides set of performance testing evaluated on binders a regular asphalt concrete mix of 0-11mm grading has been designed. For each type of additive used in 50/70 distilled bitumen assessment has been done to determine the appropriate reduced manufacturing temperature at which later test specimens have been produced. In general for most of the used additives the manufacturing temperature was reduced by 20-30°C. On the mixes traditional as well as performance based characteristics have been evaluated and compared. Gained results are presented in the paper.

Keywords: warm asphalt mix, low-viscosity binders, waxes, tensides, dynamic viscosity, durability, permanent deformations, crack propagation, flexural strength

1 Introduction

Reducing manufacturing temperatures of bituminous mixtures has become one of the main preoccupations in the road-industry worldwide. Warm mix asphalt technologies (WMA) allow the producers of asphalt pavement materials to lower the manufacturing temperatures at which the material is mixed and paved on the road. During the last decade warm mix technologies have rapidly developed especially in the USA. Recent overview of American specifications, ongoing research and the state of the practice is for example presented in [1]. Overview of 24 different WMA technologies used in USA is on a web site devoted to WMA (http://www.warmmixasphalt.com/WmaTechnologies.aspx).
WMA are used often also in some European countries for example in France and Germany. European Asphalt Pavement Association (EAPA) promotes the use of WMA. The position paper of EAPA on WMA with references dated 2010 is on its web page (www.eapa.org). There is also a practical guide on WMA on the web site of the German Asphalt Association (DAV; www.asphalt.de) dated 2009. German Road and Transportation Research Association FGSV

published in 2011 the basic guidelines related to WMA application "Merkblatt für Temperaturabsenkung von Asphalt".
There has been also an intensive research on WMA in the Czech Republic at the Czech Technical University in Prague (CTU) and Technical University of Brno (VUT) especially in the framework of the research seven years lasting virtual research centre CIDEAS focused on advanced structures and designs. This led to the publication of results on the national and international conferences [2, 3]. Some research on various WMAs has been also carried out by the Eurovia Services either within CIDEAS or as its own development realised in co-operation with the specialists of the Research Centre of Eurovia in France [4].
The traditional compacted asphalt mixes which constitute about 80-90 % of all pavement structures in the Czech Republic as well as a number of other European countries are produced, paved and compacted – depending on the bituminous binder applied – under temperatures ranging from 140 °C to 180 °C (for mastic asphalts which are not classified as compacted asphalt layers, the temperatures even exceed 240 °C). Such temperatures are required to achieve a sufficient balance between the necessary viscosity of the bituminous binder needed to ensure sufficient coating of the aggregate grains, good workability during paving and compaction, fast onset of mechanical strength and durability in asphalt pavement structure exposition to repetitive stress by transport and changing weather conditions. As a rule, traditional technological methods are energy-intensive and constitute sources of emissions of greenhouse gases as well as other hydrocarbon compounds and vapours emitted. Although, according to the latest IARC study, the vapours have no carcinogenic effects they are still widely discussed and monitored. Any reduction of the working temperature required reduces both the vapour concentration and the qualitative composition.
Possibilities for reducing the power consumption may not be searched for solely in material base optimisation and application of suitable additives or technical methods. In the case of asphalt mixes, the entire production process in the mixing plants must be improved; this involves as consistent insulation of the piping, storage tanks and the mixing equipment as possible as well as reducing the binder and resulting mix storage time; ultimately, this impacts a number of other factors. Similarly, considerable energy savings can be achieved through appropriate ways of aggregate storage (covered depositories with reduced moisture access). Due to that, a wholesome systemic perspective is necessary, i.e. perception of the production, transport, paving and compaction as a single system of partial processes where an effort is made to reduce its overall energy consumption. Undoubtedly, a related measure is a gradual transition to such power sources or power generation methods that will reduce in decrease of the greenhouse gas emissions generated – primarily of CO_2 – over time. Asphalt mix technology optimisation is a combination of both systems, i.e. the part depending on the product itself (asphalt mixes) and the part depending on the production facilities and machinery.
From the point of view of WMA design, a number of additives or certain technological procedures may be applied. The additives used to reduce the production or paving temperature for asphalt mixes can be classified in several basic categories:
· Paraffin-type additives – operating on the basis of binder viscosity reduction when heated above the softening point of the binder itself while improving asphalt stiffness under temperatures below the softening point of the additive itself.
· Surfactants – the primary effect of these liquid additives is reducing the surface tension at the meeting of the bitumen and aggregate phases, thus improving the aggregate coating by the binder. The reduced surface tension which is achieved through temperature increases in common hot road asphalt results not only in improved adhesion but also improved compactability of the asphalt mix which allows decreasing the temperature employed during paving and compaction.
· Mineral additives applying the micro-foam process – the addition of such additives during the mixing of the asphalt mix (the most commonly used ones are zeolites) results in the formation of very fine foam that contains micropores and improves workability of the mix.

With respect to the nature of the additive, the application must take into consideration the time restrictions of its effect.
- Foamed bitumen mix process – in a special mixing device, water in the form of steam and air are injected in the hot binder through jets under high pressure. This creates foamed bitumen which, depending on the quantity of water, increases in volume several fold and facilitates the coating of the finer aggregate grains in particular; this in turn provides for a relatively high-quality mortar binding the larger aggregate grains to form.

2 Bituminous binders used for WMA designs

As part of research activities within the ongoing cooperation between CTU Prague and contractors/industry several variants of low-viscosity bituminous binders have been designed and experimentally prepared. In parallel three low-viscosity binders have been tested, which have been prepared by the refinery PARAMO and The Research Institute for Inorganic Chemistry. Binders labeled NV40, NV41 and 407 were delivered for this testing. These binders contained different content of additives based on tensides and synthetic waxes. For better comparison in case of some tests done with these three binders bitumen 50/70 with 1 % of IterLow additive and low-viscosity binder 50/70 with 3 % FT paraffin is used. For these the low-viscosity binders the origin of distilled bitumen was different from the bitumen used in delivered binders NV40, NV41 and 407. The mixing of all low-viscosity binders prepared by CTU was done at the temperature of 150-155 °C by using a laboratory mixing unit IKA and mixing speed of approx. 300 rpm. For assessed bituminous binders following tests have been done:
- softening point determination by means of the ring and ball method (EN 1427);
- determination of needle penetration under 25°C (EN 1426);
- dynamic viscosity determination (EN 13302)
- determination of the complex shear modulus G* and phase angle δ (geometry PP25, τ=2000 Pa, temperature interval: 20-60°C, frequency interval 0.1-10 Hz);
- frequency sweep for G* and δ with subsequent plotting of the master curve for the reference temperature of 20°C;
- multiple stress creep recovery test

Further are presented only some of the obtained test results.

Table 1 Basic empirical characteristics of bituminous binders

Bitumen	Penetration (0,1 mm)	Softening point (°C)	Penetration index (-)
50/70 (samp. 1)	64	49.2	-0.843
NV40	64	49.1	-0.842
NV41	61	50.8	-0.537
50/70 (samp. 2)	73	47.1	-1.058
50/70 IT	70	47.0	-1.229
50/70 FTP	48	75.4	3.49

As is obvious from the results of empirical testing, low-viscosity type binders achieve values comparable to the reference binder. As an interesting benchmark, binder 50/70 by different producers is added; this was applied to prepare the options with IterLow and FTP. A minimal to zero effect of the surfactant IterLow is seen in contrast to the well-known, significant effect of FT paraffin as known from both literature and practice. Particularly with respect to the results indicated below, the evaluation of the penetration index (PI) is interesting. Here, it can be seen that the binders prepared within the framework of low-viscosity bitumen development in the PARAMO refinery demonstrate a minimum PI change. These are binders with

lower thermal sensitivity; from the point of view of resistance to deformation they are rather average bituminous binders. In the case of the second set of low-viscosity binders related to the 50/70 sample 2, it is obvious that the application of IterLow results in further reduction of the thermal sensitivity of the bituminous binder to the detriment of the ability to show sufficient resistance to deformation. Contrastingly, the binder with FT paraffin behaves in the opposite manner; its resistance to deformation (stiffness) improves substantially; however, this is likely to mean a higher thermal sensitivity at the same time, particularly in the low temperature area. No measurements were taken for binder marked 407 with respect to the fact that the primary purpose involved an application of the binder in the asphalt mix. Due to the limited quantity, there was not enough material to test the binder in itself.
In the case of dynamic viscosity measurements taken, conversion to the shear rate 20 rpm, which is considered a standard parameter in USA. The results are presented in Table 2. It is more than obvious that both options prepared by PARAMO have an effect on viscosity reduction of the bituminous binder. This is also confirmed by the chart below. From our point of view, the low values of the binder by a different producer are affected by the circumstance that the parameters of the binder seem rather like those of bitumen 70/100 which should demonstrate superior viscosity parameters in general.

Table 2 Results of dynamic viscosity testing

Bitumen	Dynamic viscosity @ 6.8 s^{-1} (20 ot./min.)	
	T=135°C	T=150°C
	(Pa.s)	(Pa.s)
50/70 (sample 2)	0,5	0,3
50/70 (sample 1)	0,7	0,8
NV40	0,4	0,2
NV41	0,4	0,2

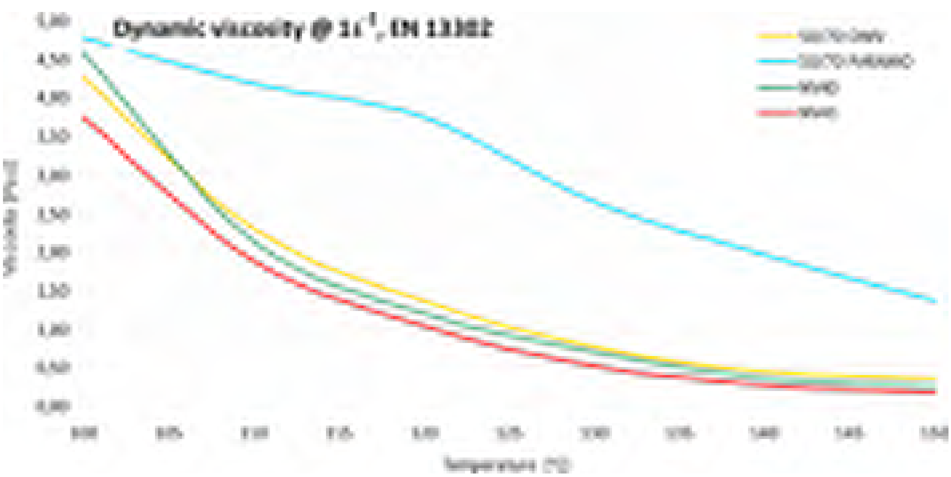

Figure 1 Dynamic viscosity (viscosity curves) at shear rate of 1 s^{-1} and temperature interval 100-150°C

From the perspective of the flow curves with shear rate of 1 s^{-1} it is obvious that the variants of experimentally prepared low-viscosity binders NV40 and NV41 under assessment reduce the viscosity value throughout the entire thermal interval. A slight decrease in the case of 100 °C is caused by its proximity to the temperature where the bituminous binder gradually changes from the liquid or quasi-liquid phase to the solid phase. Simultaneously, under this temperature there will be a loss of functionality of the additives applied to reduce temperature. In contrast, within the 105-140 °C interval dynamic viscosity of low-viscosity binders is five times lower (when compared to binder 50/70 sample 1). The other distilled bitumen compared has almost identical viscosity curve as binder NV40.

3 Assessment of asphalt mixes

In the second part of the experimental assessment asphalt concrete mixture of ACO11+ type has been selected with a design fitted for aggregates from Libodrice quarry (amfibolite). The procedure defined by Czech technical specifications TP238 for determining the proper specimen manufacturing temperature was firstly applies for the mixes with different low-viscosity binders. At the determined temperature the bulk density of compacted Marshall specimens for a WMA is equal or close to the bulk density known for a reference mix. For this reason the reference asphalt mix is produced with traditional temperature dependent on the binder used (150 °C in the case of this assessment). Then the test specimens for WMA mixes are produced for temperatures 110 °C, 120 °C, 130 °C, 140 °C and 150 °C with determination of bulk densities. By applying iterative method the comparable bulk density to reference mix is searched and the temperature at which such temperature is gained, determined. Nevertheless, based on the results received it was again demonstrated that this approach is at least arguable and during next revisions of the technical specifications TP238 this approach should be critically reviewed. It might be applicable to mixes where paraffins or waxes are used, but definitely not for mixes where different surfactants are applied. Presently it is not fully obvious, what approach could suitably replace the existing one (testing and assessment of dynamic viscosity, application of gyratory compactor for specimen preparation and determination of compaction effort, another procedure to be defined).
The bitumen content determined for reference mix optimization was 5.2 % by mass. The reference mix was produced at a temperature of 150°C. For remaining variants always the described temperature optimization has been done with the production of test specimens for the temperatures described above. Resulting temperature recommended for each variant of WMA mixes has been set in the range between 120 °C and 130 °C. For the mixes of second set, where additives like RH, IterLow, FTP, Zycotherm and industrially ready-to-use bitumen ECO2 have been applied, it is necessary to underline, that for the production of low-viscosity bituminous binder a basic bitumen was applied, which officially was declared as 50/70 pen grade, nevertheless by its parameters rather fulfilled criteria of a 70/100 bitumen. This fact has to be kept in mind especially during the later evaluation and discussion of gained results. For asphalt mixes with selected referential temperature subsequently selected tests have been done:
· determination of bulk density of compacted test specimen;
· determination of maximum density and void content of an asphalt mix;
· determination of water susceptibility by indirect tensile strength test (ITS ratio);
· determination of water susceptibility by applying one frost cycle and assessment of indirect tensile strength (ITSR_f);
· resistance to permanent deformations by wheel tracking test;
· stiffness modulus assessment (IT-CY test method);
· determination of flexural strength at -5° C;
· determination of asphalt mix resistance to crack propagation at 0 °C.

Results of done specimens compaction temperature optimization and the progress of basic empirical results as well as functional characteristics are presented further in this paper.

Table 3 Basic characteristics of assessed warm asphalt mixes

Mix variant	Compaction temperature (°C)	Bulk density (g/cm³)	Maximum density (g/cm³)	Voids (%)
ACO 11+ REF 50/70 (sam. 1)	150	2,673	2,745	2,61
ACO 11+ NV40	120	2,661	2,740	2,88
ACO 11+ NV41	130	2,639	2,747	3,93
ACO 11+ 407	120	2,676	2,740	4,16
ACO 11+ 3% FTP	130	2,584	2,722	5,07
ACO 11+ 3% RH	120	2,620	2,722	3,76
ACO 11+ 1% IT	130	2,570	2,722	5,57
ACO 11+ 0,1% Zycotherm	150	2,621	2,722	3,71
ACO 11+ ECO^2 (130°C)	130	2,520	2,657	5,16
ACO 11+ ECO^2 (120°C)	120	2,537	2,657	4,53

Out of the basic characteristics listed (volumetric weight and void content) it is the void content that is crucial. Here, it was discovered that with the exception of the mix with 3 % FTP, all of the remaining mixes meet the requirement for ACO 11+ mix type stipulated by CSN EN 13108-1 (2.5-4.5 %-vol.). It is obvious that the mix with binder NV41 where a lesser temperature reduction was recommended demonstrates a greater void content increase in comparison to the reference mix. The fact rather supports prioritisation of the option with binder NV40. At the same time, it should be taken into consideration whether, in case of binder 407, the limit temperature should be 130 °C. In such a case, better compaction and lower void content in the test specimen might probably be achieved.
In the case of the second set of experimental mixes, it can be noticed that from the perspective of void content, additive RH records very good results under compaction temperature 120 °C. As long as, chemically, the binder is of a type similar to FTP, a greater potential is obvious in this regard; superior compaction is achieved even under lower temperatures. In the case of Zycotherm, the test specimens were not produced under lower working temperature since the verification of possible reduction of the working temperature as specified in the preliminary TP238 did not correspond with this option. Based on the findings, we can probably consider a maximum working temperature reduction to 135-140 °C which, however, will result in a slightly increased void content. An interesting trend appears in the case of the industrially manufactured low-viscosity binder marked ECO^2. It is shown that specimens prepared under the temperature of 120 °C achieved lower values when compared to a working temperature increased by 10 °C.
During the ITSR test which is currently the determining factor from the point of view of asphalt mix durability assessment, test specimens are prepared by compacting (2x25 blows by impact compactor). The specimens are divided into two groups – one left at room temperature and the other soaked in water and, subsequently, tempered in a water bath for 3 days at 40°C. Then, an indirect tensile strength is taken and a strength ratio is determined; according to CSN EN 13108-1 its value should be below 70 % for an ACO11+ type mix. In the case of the ITSR_f test, the aforementioned procedure is modified in compliance with the American methodology (AASHTO 283T) and the specimens are water-soaked for shorter periods but, afterwards, placed in a freezer at approx. -18°C for 16 hours. Then, the specimens are immediately placed in a water bath at 60°C for 24 hours. The method should result in the specimens being stressed more by various effects and, therefore, lower indirect tensile strength (and a poorer strength ratio value) is expected. In relation to the modified ITSR indicator, CTU proposed a 10 % lower permitted threshold in relation to the standard ITSR indicator threshold to provide for the effect of frost in particular.

Table 4 Assessment of mix water susceptibility

Mix variant	ITS_{dry} (MPa)	ITSR	ITSR_f	Modulus ratio
ACO 11+ REF 50/70 (sam. 1)	1,91	0,90	0,83	0,78
ACO 11+ NV40	1,68	1,08	1,03	1,12
ACO 11+ NV41	1,65	1,06	1,04	1,05
ACO 11+ 407	1,45	1,00	-	1,02
ACO 11+ 3% FTP	1,37	1,13	0,67	1,58
ACO 11+ 3% RH	1,05	1,23	0,90	1,22
ACO 11+ 1% IT	1,59	1,10	1,07	1,08
ACO 11+ 0,1% Zycotherm	1,70	1,08	0,77	0,93
ACO 11+ ECO2	–	–	–	–

Last but not least, besides ITSR and flexural strength of specimens not soaked in water or frozen, the calculated value of the elasticity modulus can be determined (this is easy to calculate from the indirect tensile strength and the horizontal deformation values). No requirements are stipulated for this parameter; it is only quoted as an informative value. It may be used as a guide for the assessment of material stiffness when the test specimen reaches destruction (collapse). In the case of using the flexibility modulus characteristics to determine the ratio of the proportion between the dry and soaked specimens, this is an alternative to the ITSR characteristic.
From the perspective of the test of asphalt mix resistance to permanent deformation, standard CSN EN 13108-1 stipulates a requirement for PRD_{AIR} and WTS_{AIR} solely in relation to mixes ACO 11S (5.0% and 0.07 mm, respectively). For “+“ mixes, the value is merely declared; no limit value is determined. Class “S” is used for roads with very high intensity of heavy loaded vehicles. As is obvious from the measurements taken, the mixes assessed meet the requirement of the “S” class criterion; only the mix with 1 % IterLow significantly exceeds the WTS_{AIR} indicator. When the mixes are compared to one another, we can see that there are no material influences from the point of view of the first indicator. This also applies to the second indicator. The improved value of PRD_{AIR} in the mix with NV41 and, contrastingly, deterioration of the WTS_{AIR} indicator which suggests that with continuing stress, the depth of track increases more rapidly, are interesting. What is important is that no negative impact on deformation characteristics was recorded with a lower production temperature. Even in the case of the mix with 3 % FTP where a higher void content (and, therefore, a higher risk of additional compaction) was noticed, the mix shows very good characteristics in relation to permanent deformation. Limit results were achieved by the mix with IterLow; in the case of the WTS_{AIR} indicator, this version failed to meet the standardised requirements. An increased value of the parameter is indicated also in the case of Zycotherm.

Table 5 Results of wheel tracking test (small device, air bath at 50 °C)

Mix variant	Relative rut depth after 5 000 cycles v mm – PRD_{AIR} (%)	Increment of rut depth WTS_{AIR} (mm)
ACO 11+ REF 50/70 (sam. 1)	2.2	0.026
ACO 11+ NV40	2.3	0.030
ACO 11+ NV41	1.9	0.034
ACO 11+ 407	n.a.	n.a.
ACO 11+ 3% FTP	2.0	0.019
ACO 11+ 1% IT	4.5	0.074
ACO11+ 0,1% ZT	2.6	0.048

The last group of testing presented here is characteristics that describe the asphalt mix behaviour under low temperatures while it is technically possible to assess the behaviour under temperatures ranging from 0 °C to -10 °C. The method of determining flexural strength is based on a three-point beam test when two stress levels are selected (50 mm/min and 1.25 mm/min), as specified in TP151. At present, the test is only required for HMAC type mixes with a minimum strength criterion of 6 MPa for the lower stress speed and temperature of -5 °C. Flexural strength determination under the higher speed can be perceived as an indicator of potential problems with cracking under low temperatures. The other test method applied to compare the individual mix versions is the asphalt mix resistance to crack propagation in compliance with CSN EN 12697-44 which is conducted on semi-cylindrical test specimens by means of a three-point test with weakened profile (a groove of 10 mm depth) at the point of stress by a constant force. When compared to the flexural strength test, the latter method appears to be a more appropriate method to assess actual crack propagation of the mix.

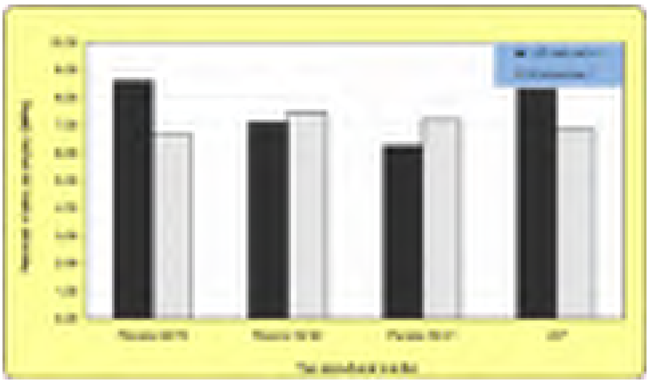

Figure 2 Results of flexural strength test, set I

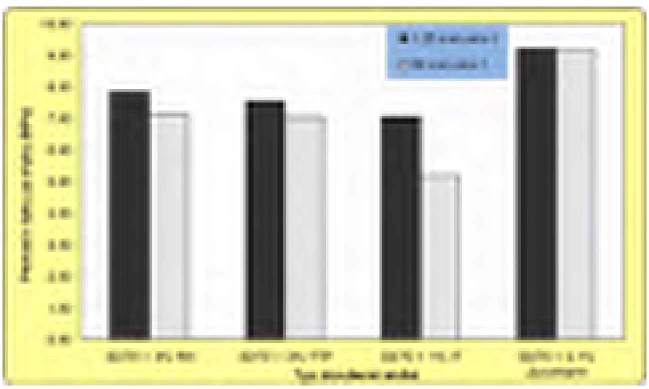

Figure 3 Results of flexural strength test, set II

If, in accordance with TP151, we restrict ourselves to the values obtained for the lower stress speed, it is obvious that none of the low-temperature mixes from the first set of mixes assessed reaches the maximum detected for the reference mix although the values exceed the 6 MPa limit. It is also shown that from the perspective of the parameter of mix behaviour under low temperatures the best behaviour was detected for the asphalt mix with binder 407. When binders NV40 and NV41 are compared, the former is more appropriate. The aforementioned statement is very well confirmed by the crack propagation resistance test summarised in Table

6. The crucial indicator therein is the fracture toughness $K_{ic,i}$ where, again, the highest values were recorded by the reference mix and, from the point of view of low temperature asphalt mixes, the option with binder 407 appears to be the most suitable mix.

Table 6 Results of resistance of asphalt mix against crack propagation

Mix variant	Strain $\varepsilon_{max,i}$ (%)	Fractural toughness $K_{ic,i}$ (N/mm$^{3/2}$)	Stress $\sigma_{max,i}$ (MPa)
ACO 11+ REF 50/70 (sam. 1)	1,30	40,80	5,50
ACO 11+ Paramo NV40	1,43	36,70	4,95
ACO 11+ Paramo NV41	1,23	34,84	4,68
ACO 11+ 407	1,23	37,69	4,54
ACO 11+ 3% RH	1,38	35,49	4,78
ACO 11+ 3% FTP	1,33	36,06	4,85
ACO 11+ 1% IT	1,23	39,30	5,31
ACO 11+ 0,1% Zycotherm	1,23	33,69	4,54
ACO11+ ECO^2 (130°C)	1,00	31,86	4,28
ACO11+ ECO^2 (120°C)	2,71	27,41	3,69

Regarding the second set of experimentally evaluated mixes, it is obvious that the best values of flexural strength were achieved by the asphalt mix with bitumen where Zycotherm was applied as an additive. In this case, if compared to the reference mix, a better flexural strength value was achieved. Good results were recorded also for the application of additive RH. An interesting finding was noticed from the perspective of strength values of the asphalt mix with Zycotherm where basically the same values were reached for both speeds while the values even more significantly exceed the results of the reference asphalt mix. In contrast to that, the results obtained in the asphalt mix resistance to crack propagation test do not relate to the flexural strength test values. Particularly if the result obtained for the asphalt mix with Zycotherm is assessed, it is obvious that the values obtained from the semi-cylindrical specimens are the worst in the comparison of both mix sets while the result is confirmed by the significantly higher deformation generated. A similarly contradictory result arises from a comparison of the values for the asphalt mix with IterLow; where in the case of flexural strength determination the mix recorded the lowest values amongst the second set of mixes while a value comparable to the reference mix with a lower deformation level was achieved from the perspective of fracture toughness. These findings thus open a whole area for discussion concerning the possibility of mutual substitutability of the two aforementioned test methods. Looking at the results of both tests, we can cautiously formulate a hypothesis on a possible link between the results of both methods.

4 Conclusion

As is obvious from the results presented, the most suitable solution for low-viscosity bituminous binders cannot be determined with absolute clarity. The greatest working temperature reduction is achieved by options with NV40 and 407 in the first set while, in the second case, a slightly higher void content is reached. Nevertheless, the mix with this binder still scores highest in relation to stiffness and low-temperature properties characteristics. No test of resistance against permanent deformation was taken for this mix with respect to the limited quantity of the bituminous binder. The option with binder NV40 achieved balanced results. In both cases, further improvements can be expected if the asphalt mix, or test specimen production takes place under the temperature of 130 °C. Due to the above mentioned, we recommend paying attention and possibly undertaking further optimisation measures in relation to these two binders. At the same time, we emphasise that the experimental results be

confirmed by a test section, or possibly, before a test section is completed, by experimental methods with another asphalt mix type.
In the case of the second set of mixes assessed, no clearly best option can be determined. From the point of view of low-temperature characteristics, the options with additives RH and FTP have even properties from the perspective of flexural strength and crack propagation. With resistance against crack propagation, the mix with IterLow scores best while the same binder causes the poorest result from the flexural strength point of view although all of the options meet the minimum requirement for VMT mixes according to TP151 in absolute values. The mixes with FTP demonstrate good stiffness modulus values at 15 °C; in the case of binder ECO^2 the result is similarly satisfactory. In contrast to that, the stiffness values were too low for the mixes with Zycotherm or RH. If a separate evaluation of the result for the parameter of resistance against permanent deformation is made the higher values in the case of the mix with IterLow are noticed. This fact is principally very well complemented by the PI values detected. Nevertheless, it should be considered whether test specimen preparation under 120 °C or even 110 °C would yield similarly satisfactory results (the IterLow manufacturer even mentions a possible working temperature reduction to the 90 °C level). In such a case, the binder would present a well-balanced compromise between the individual characteristics. With respect to additive RH, the question is how the benefits or shortcomings of its application might be compensated by the price. This is an additive of Chinese origin and, therefore, a more aggressive pricing attitude can be expected. From the perspective of technical data, the mix was assessed under 120 °C while thermal optimisation according to TP238 would probably rather suggest 130 °C. Tests of the asphalt mix with FTP proved the traditional findings that can be tracked in other studies conducted as well. The issue of behaviour of the bituminous binder with FTP under low temperatures can probably be tackled in the future solely by combined modification with polymers.
It should also be emphasised that no problem with durability as indicated by ITSR was detected for any of the versions examined. In general, this area is critically researched in the long term in many cases as a number of studies available and verification testing undertaken suggest that low-temperature asphalt mixes might be highly susceptible to ITSR parameter deterioration. This fact is indicated, solely in the second set of mixes under examination, by the asphalt mix with the FTP-modified binder when a single freezing cycle method is applied. Such saturation and stressing of test specimens results in a significant decrease of the ITS values.

Acknowledgments

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Reference

[1] Barros, C.: Warm Mix Asphalt State of the Practice. 51st Idaho Asphalt Conference October, 2011.

[2] Valentin, J., Mondschein, P., Varaus, M., Hýzl, P.: Stiffness and Low-Temperature Behavior of Selected Warm Asphalt Mixes. Proceedings of 11th International Conference on Asphalt Pavements (ISAP), Nagoya, Japan, 2010.

[3] Valentin, J., et al.: Performance characteristics of selected warm mix asphalt, (in Czech) Conf. AV 11, November 2011.

[4] Delfosse, F., Fiedler, J., Kašpar, J., Sýkora, M., Bureš, P.: Warm mixes at Eurovia, Bratislava 2011.

[5] Delfosse, F., Giorgi, C., et al.: Development of the Evotherm DAT® process in Europe. Collaboration between Eurovia and Meadwestvaco, In: Proceedings of IRF, Lisbon 2010. (http://www.meadwestvaco.com/mwv/groups/content/documents/document/mwv011338.pdf)

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VIASPHALT BT®, THE MASTIC ASPHALT "LOW" AND "VERY LOW" TEMPERATURE

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Abstract

Mastic asphalt is a common material mainly use for waterproofing works (bridge decks), for pedestrian paths, car parks but also for road wearing courses. This material needs high temperatures in order to aloud correct laying conditions. The European REACH regulation (Registration Evaluation and Authorization of Chemicals) started to be implemented in Europe in 2007. The application to bitumen on the 1st of December 2010 had a critical effect of the development of warm solutions for mastic asphalts. Starting with experimental products, the French companies had to move to industrial solutions by producing and laying mastic asphalts under the temperature of 200°C. This temperature corresponds to the maximum temperature to use the bitumen, as it was defined by the producers. Characteristics of the products are defined according to the standard EN 13108-6. In this context the Eurovia Group developed a range of mastic asphalt products corresponding to the 2 new types (between 180°C and 200°C and under 180°C). The technic is very innovative and based on the use of a single additive which is renewable and which is very easily implemented on the plant. These new mastic asphalts are produced around 170 / 180 °C and are laid on the jobsite between 160°C and 180°C. This is a real drop by comparison with traditional mastic asphalt which are produced and laid around 220°C to 240°C. The aim of the presentation is to detail the formulation process, the performances and the follow up of these new products as well as their implementations in France, Czech Republic and CROATIA by the company TEGRA.

Keywords: warm asphalt, health care, condition of laying, European regulation, innovation

1 Introduction

The current mastic asphalt is defined by the following way. It is a mixture of chippings, sand, filler and bitumen the proportions of which are adjusted in order to obtain the flowing characteristic. Currently the mastic asphalts represent 220,000 tons laid on the French territory [1] and 1 million tons in Europe. The common proportions for a road mastic asphalt are:
- Chippings 30%;
- Sand 34%;
- Filler 28%;
- Bitumen 8%.

In general, the components are mixed in the plant with the quick mixer, and then they stay, for homogenization, in the pug mill during some hours. The mastic asphalt is then transported on the jobsite by trucks called transport mastic asphalt pug mills, thermo regulated, horizontal and vertical and with a capacity of 8 to 20 tons. The laying is, the most of the time, carried out manually by taking the mastic asphalt in wooden buckets and by laying it with a wooden squeegee (trowel).

For the jobsites that need an application on a continuous big surface, mechanized means are used. There are specific pavers (pendulum beam) front fed by a mobile plug mill. For a longtime, the mastic asphalts are laid in the following sectors: waterproofing, footpath, industrial soil and roads. The production and the laying of mastic asphalts were carried out traditionally with temperatures reaching 240 to 250°C.
As for the asphalt concretes, one of the main research axes regarding these products is the decrease of their production and laying temperature. The field of mastic asphalt suffered a profound and quick modification in 2010 following to the coming into force of a new regulation. The implementation of this new regulation called REACH [2] began in June 2007. The application with bitumen on 1st December 2010 had an accelerating effect on the elaboration of the warm mixes. At the experimental stage, the companies were forced to move to the industrial stage while producing and laying the products at a maximum temperature of 200°C. This temperature corresponds to the maximum temperature of the use of the binder defined by the bitumen producers. The bitumen of direct distillation with the CAS number (unique registration number at the data bank of chemical abstract service) 8052-42-4 had the deadline of registration at the ECHA [3] on 30/11/2010. This registration in REACH, expressed on behalf of the Oil Companies by Europeans Unions EUROBITUME and CONCAWE, indicates via safety data sheets, the conditions of use and the use defined by the producers of bitumen, particularly the maximum temperatures to not exceed regarding health and safety rules. The safety data sheet of the bitumen commonly used for mastic asphalt systematically indicates a heating temperature up to 200°C or the temperature of the flash point reduced by 30°C, that means 190°C or 200°C depending on the source of bitumen.
The consequence of the bitumen registration in REACH was a production and laying of a mastic asphalt at temperatures below 200°C. In March 2012 the French "Office des Asphaltes" which is the Union of mastic asphalt Industries in France decides to define again the terminology of mastic asphalts while creating 3 classes of mastic asphalt form its general definition:
- traditional mastic asphalt: mastic asphalt, produced, transported and laid at a temperature above 200°C;
- low temperature mastic asphalt (BT): mastic asphalt, produced, transported and laid at a temperature between 180°C and 200°C;
- very low temperature mastic asphalt (TBT): mastic asphalt, produced, transported and laid at temperatures below 180°C.

In the 3 cases the mixing characteristics are defined by the standard NF EN 13108-6.

2 EUROVIA – the VIASPHALT® BT

2.1 Development and characteristics

For more than 10 years Eurovia has worked to decrease the temperatures of mastic asphalts. This step and the associated researches meet the environmental and health policies of the group. The research center of Eurovia elaborated a process allowing to decrease very significantly the production and laying temperatures of these mastic asphalts for the most common applications. This temperature lowering has numerous advantages, on the technical level (important decrease of the shrinkage phenomenon), economical (low extra costs), environmental and health level (energy consumption and almost no more fumes while applying).
The process is extremely innovative by its simplicity (unique additive), by the renewable character of the additive (in comparison to those currently used in the profession), by a production mode easily transposable in the batch plants. The decrease of the temperature between 60°C and 70°C allow to produce the mastic asphalts at 170°C / 180°C. Regarding the nomenclature of the office of mastic asphalts Eurovia carries out in the majority of the cases mastic

asphalts TBT. The process is subject to a trademark in Europe and in a number of countries all over the world. It contributed to the new product, the VIASPHALT® BT.
The principle is based on the use of an additive allowing to liquefy the mastic asphalt and to modify the pseudo viscosity of the mixture aggregates / bitumen at high temperatures (> 100°C) without changing the performances of the mastic asphalt below 80°C, or to improve them for certain ones.
This additive coming from the agro-resource, thus renewable is characterized by a melting point of 90°C. Its production simplicity makes the supply reliable and regular, furthermore its packaging in thermo fusible bags or big bags and its handling is simple. Before 2010, several experimental jobsites were carried out in France in order to validate the process, to make sustainable innovation.

2.2 VIASPHALT BT®, key benefits

2.2.1 Mix design

The mix design is the same as for hot mixes, as a part of bitumen is replaced by an additive. The process is then simple but keeps all known difficulties of the mastic asphalt mix design, workability; shear resistance (Fig.1). In the majority of the cases, the elaboration in the laboratory is not only the feasibility; validation and industrial adjustment are consequently necessary.

Figure 1 Machine to measure the indentation according to the standards EN 12697-20 and EN 12697-21

2.2.2 Technical characteristics

They are maintained or improved for certain regarding the traditional mastic asphalt. A particular characteristic of VIASPHALT BT is its lower shrinkage sensitivity. This one measured in the laboratory following the operation mode used in France allows us to verify this excellent behaviour, as shown on Table 1.

Table 1 Comparative shrinkage measurements

	Traditional mastic asphalt (footpath)	Viasphalt® BT (footpath)
Shrinkage [mm]	0.20	0.07

2.2.3 Economical
Whatever the process used to decrease the temperature of mastic asphalts, systematically there is an extra cost non-negligible. With an increase of 15% in comparison to the traditional mastic asphalt, the VIASPHALT® BT is very competitive on the market of the low temperature mastic asphalt.
Note that the reduction of energy consumption in production does not present an economical advantage, as this one is largely absorbed by the cost of the additive. This is a very important point for the development of the process, adding of the additive is carried out by a thermo fusible bag or by an automated system in the asked quantity (see Fig. 2). The production is possible on every batch plant without changing significantly the production conditions

Figure 2 Automated feeding of the additive (Plant CIFA Mitry-Mory, France)

3 Situation in 2013, EUROVIA, France and abroad

In France, the production of mastic asphalts BT and TBT covers at least 80% of the tonnages carried out by the companies that are members of the office of the mastic asphalts. Since December 2010, Eurovia has laid low temperature mastic asphalts for the quasi totality of its mastic asphalt activity. (~98%). There are still some jobsites carried out with hot mixes on request of the client. These jobsites concern generally the waterproofing on bridges, sector in which the mastic asphalts BT show satisfied results. However there are still some reticences on behalf of certain clients because of a lack of technical information. The same for the most jobsites with clear or colored mastic asphalts based on a clear binder that are still carried out with the traditional process.
In other countries mainly in Europe, there are less feedback and communications on the use and development of these new mastic asphalts. Eurovia distributes its process within its subsidiaries. The first jobsites were carried out in the Czech Republic, in Prague in 2012.
In order to develop this process in the Czech Republic in the subsidiary EUROVIA CZ, the company had to use all its know-how from France on low temperature mastic asphalts. Thus, the first step of this technology transfer was to choose carefully the components and to establish

a mix design with the cooperation between the laboratories of Mérignac and Prague. The second step was to launch a first satisfactory industrial production (see Fig.3).

Figure 3 Resurfacing TRAMWAY (PRAGUE 2012)

In 2013, the subsidiary TEGRA in Croatia carried out its first experimentation. A similar deployment was undertaken by entities of the company to take full advantage of the experience gained in France. The application fields sighted by the Croatian subsidiary are bicycle paths or footpaths, roads with light traffic and parking spaces in commercial areas. The same requirements regarding indentation are expected in Croatia. However, the technical export is complex, the regulation REACH is not always followed and the extra cost of production seems sometimes too high in comparison to the traditional mastic asphalts.

4 Technical report after two years of production

In the region of Paris, since 01/12/2010, Eurovia has produced and laid about 45,000 tons of “warm” mastic asphalt. Regarding the terminology of the office of mastic asphalts of March 2012, the big majority was carried out in TBT that means below 180°C. For the transport, the horizontal and vertical pug mills are now systematically calibrated in order to guarantee the announced temperatures. Furthermore the maximum heating temperature is fixed at 190°C, there is no action possible on the temperature to catch up a lack of workability and there is no scope of action to adapt production, transport and laying hazards.
The conditions of production, storage and putting in pug mills and transport became very precise and very qualified. The complexity caused by the decrease of the temperature and the important increase of the viscosity of the binder asked important industrial follow-up. One discovers then a difficulty to maintain the indentation level of the production up to the last ton of the pug mill, hence the difficulty to produce daily regular mastic asphalts. Difficulties that are the same in the past for traditional mastic asphalts but force of habit they were less often followed by all players of the mastic asphalts.
The traditional mastic asphalt afforded some flexibilities of use for its viscosity managed by the temperatures higher than 200°C and mechanical resistances favored by the oxidation of the bitumen at a high temperature. The warm mastic asphalts kept these performances by the use of an adapted additive but the technology is more complex. On the 45,000 produced tons, beyond the follow-up of CE marking, 10% of the production and laying were subject to a high level control in the laboratory and on the jobsite (workability, indentation, complete analyses).
The efforts of the follow-up, the adjustments allow today to guarantee the mastic asphalts BT or TBT responding on different fundamental criteria; indentation, workability, aspect and the laying facility, compliance with thicknesses, all, in an urban environment with a quasi-

disappearance of fumes (see Fig.4). With a temperature lowering by 60°C the fumes quantity is divided by a factor of 32.

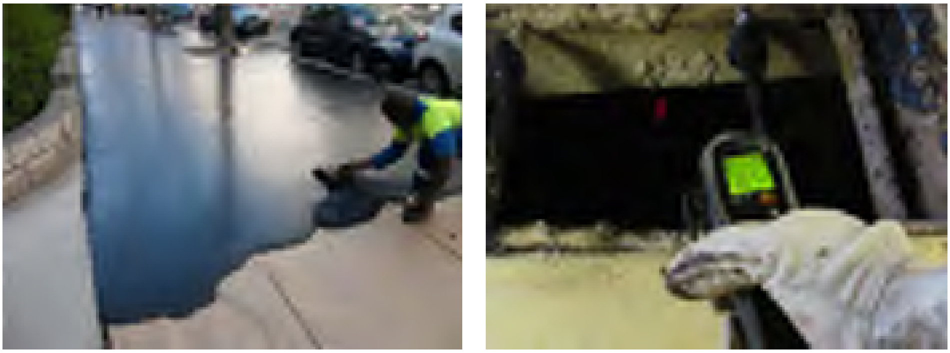

Figure 4 Jobsite VIASPHALT BT in Paris and temperature measurement

5 Health and environmental reports

The environmental report is very positive. Since 2010, Eurovia followed the impact of this production of low temperature mastic asphalt on atmospheric emissions. The main production plant of the group provided all figures, tonnage and energy consumption. The evaluations of the data in 2011 and 2012 were compared to 2010 (about 90% of hot mastic asphalt production).
The results are the following:

- the reduction of the registered energy consumption on the level of dryer is 41% when producing low temperature mastic asphalts;
- the impacts on the emissions of greenhouse gas are about – 17 %;
- the VIASPHALT BT creates a progress of a health point of view; the following observations are systematic:
  - decrease or elimination of fumes during application;
  - much less odor;
  - application comfort;
  - important reduction of the inconvenience for the residents.

6 Perspectives

For Eurovia the perspectives are multiple. The first one is to follow its process, to develop it to further improve the whole production chain, from production to laying. Then, facing the slow evolution of the profession of mastic asphalt and its working conditions, EUROVIA would like to increase its action in the field of work hardness. Increase the activity of “low temperature asphalt” in Europe is an aim. This is an ambitious program highly depending on the behaviors and ways that will be implemented in the European Community. Overcome the administrative barriers will be more complex than the technical issue.

References

[1] Figures from “Office des Asphaltes” which is the Union of mastic asphalt industries in France

[2] REACH means Registration Evaluation and Authorization of Chemicals

[3] ECHA means European Chemicals Agency

THE EFFECTS OF AGEING ON ROAD BITUMEN MODIFIED WITH THE ETHYLENE VINYL ACETATE POLYMER

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Abstract

The increase in axle loads, heavy traffic, severe climate conditions and construction failures has led to the enhancement of bitumen properties. There is a wide range of applications of polymer modified bitumens, PmBs, in road construction. Polymer modification significantly influences the rheological characteristics of the binder; therefore the use of fundamental rheological testing methods is required rather than the empirical methods.
This paper presents the polymer modification of road bitumen, BIT, with different contents of semi-crystalline thermoplastic copolymer ethylene- vinyl acetate, EVA, before and after thermo-oxidative ageing. The effects of EVA on the conventional, rheological and thermal properties of the modified bitumens were studied. The rheological characteristics of the EVA PmBs were analyzed by a dynamic shear rheometer, DSR. The primary viscoelastic functions were determined in dependence as a function of the frequency at temperatures in BIT use. The master curves were designed at a reference temperature of 30°C.
The results indicated that the thermoplastic copolymer EVA improves the viscoelastic properties of PmBs. EVA PmBs increase stiffness and elasticity at high service temperatures and low loading frequencies. The increased stiffness compared to the conventional bitumen enhances the performance characteristics of the modified bitumen and provides better protection against increased traffic loads and adverse climate conditions. The process of ageing increased the complex modulus and phase angle, and reduced temperature susceptibility. These changes were mainly due to the chemical processes, such as degradation reactions, oxidation as well as secondary processes of cross-linking which take place during the ageing of BIT and EVA PmBs. PmBs modified with EVA significantly reduce the permanent deformation under loads compared to the unmodified bitumen.

Keywords: polymer modified bitumen, rheological properties, thermal properties, thermo-oxidative ageing, permanent deformation

1 Introduction

Bitumen has been widely used for road paving applications [1]. Although it is added in a very low concentration (5 wt. %), bitumen controls the final properties and the performance of the asphalt mixture, since it is the only deformable component and forms a continuous matrix [2]. From a material engineer's point of view, bitumen can be classified as a thermoplastic material with thermally reversible properties [3]. Thus, at high temperatures, bitumen melts and solidifies so that asphalt mixtures can sustain the stress brought on by traffic. Two main limitations that were observed in pavements, yielding poor road performance, are directly associated to the bitumen matrix that surrounds the mineral aggregates at high and low temperatures. The first problem, permanent deformation (rutting), occurs at service temperatures higher than 40°C, leading to ruts in the direction of travel, and can be related to the

viscosity of the bitumen matrix. The second appears at lower temperatures (below 0°C) and can produce cracking of the road pavement, as a result of the brittle fracture of the bituminous component of the asphalt [2, 4]. Presently, the tightening of binder specifications, in order to get longer repairing periods and the reduction of the total cost of road maintenance, factors such as increased traffic, heavier loads, etc., have led to the development of new bituminous materials. The performance of road surface can be improved by modifying the bitumen with polymers, PmB [5-8]. One of the important modifiers of bitumen, which belong to the plastomers, is semi-crystalline copolymers ethylene vinyl acetate, EVA. This type of polymer is easily dispersed in and has good compatibility with generally available bitumens, as well as being thermally stable at normal mixing and handling temperatures [8, 9].
The aim of this paper is to study the effects of EVA, as a modifier of BIT in regard to the viscoelastic properties. Rheological characterization was carried out at different temperatures and frequencies before and after thermo-oxidative ageing. The frequency dependence of rheological parameters is shown by producing rheological master curves at reference temperatures of 30°C using the time-temperature superposition principle.

2 Experiment

2.1 Materials

The bitumen used in this paper was the 70/100 penetration grade base bitumen, INA Refinery Croatia, Rijeka. The polymer used as a modifier is a semi-crystalline copolymer, ethylene vinyl acetate, EVA, commercial grade Elvax 265, containing 28 wt. % of vinyl acetate with a melt index of 6, manufactured by DuPont, USA.

2.2 Sample preparation

EVA PmBs was prepared by melt blending with a high shear mixer, Silverson L4R. The EVA copolymer was mixed with the bitumen by using a preparation method developed in the laboratory to maximize the rheological properties and to minimize bitumen degradation. 500 g of bitumen was added to the container and heated to 160°C and continuously stirred for about 2h to obtain homogeneity and then was poured into 1 L aluminum cans. The cans of bitumen were then heated to 180-185°C and stirred for 10 min before adding a polymer. Upon reaching 180°C, a weighed amount of polymer was slowly added to the bitumen. The EVA contents used were 3, 4, 5 and 7 % by weight of blend. Mixing then continued at 180°C for 4h to produce homogeneous mixtures. After completion, the EVA PmBs were removed from the aluminum cans and divided into small containers covered with aluminum foil and stored for further testing at ambient temperature.

3 Measurements

3.1 Conventional tests

The traditional, conventional tests, such as penetration (HRN EN 1426), softening point (HRN EN 1427) and elastic recovery (HRN EN 13398), were first conducted for bitumen and EVA PmBs samples according to standards.

3.2 Rheological measurements

The rheological properties of neat bitumen and EVA modified bitumen were measured by a dynamic shear rheometer (DSR), MCR 301, Anton Paar. The tests were conducted over a range of temperatures and loading frequencies in order to provide a complete characterization of the viscoelastic properties of the binder [7, 8, 10, 11]. The DSR tests were performed under controlled strain loading conditions using frequency sweep tests. The frequency sweep was applied over the range from 0.1 to 100 rad/s at temperatures between -5°C and 60 °C. The strain was kept low enough so that all tests were performed within the linear viscoelastic range (LVE) [10, 12]. The tests at temperatures between -5°C and 30°C were carried out with a parallel plate testing geometry of 8 mm diameter and 2 mm gap, and from 30°C to 60°C were done with a parallel plate testing geometry of 25 mm diameter and 1 mm gap. To provide a more profound insight into rheological properties, the critical temperature without permanent deformation (rutting) was determined according to the SHRP [9, 12, 13]. The critical SHRP temperature, T_c/°C, is the temperature at which $G^*/\sin\delta \geq 1$ kPa before ageing and $G^*/\sin\delta \geq 2.2$ kPa after ageing at a frequency of 10 rad/s. The critical temperature was determined automatically by the DSR software.

3.3 Ageing procedure

Thermo-oxidative ageing of base bitumen and EVA PmBs was performed using the Rolling Thin Film Oven Test, RTFOT, according to ASTM D 2872. The bitumen and PmBs were exposed to elevated temperatures (163°C) to simulate the conditions during the production, mixing and laying of asphalt mixtures.

3.4 Differential scanning calorimetry, DSC

In an attempt to quantify the ageing mechanism associated with RTFOT ageing of the EVA PmBs, DSC thermal behavior was studied with a Mettler Toledo DSC 822e in a nitrogen atmosphere at a heating rate of 10°C/min. In the first heating and cooling scans, the samples were heated from 25°C to 150°C and held at that temperature for 10 min in order to eliminate any previous thermal history. Then, the samples were cooled with liquid nitrogen from 150°C to -120°C.

4 Results and discussion

The DSC studies have been carried out to understand the thermal behavior of the EVA PmBs before and after RTFOT ageing and the results are presented in Table 1. Changes of the super molecular structure with the addition of the EVA polymer were followed. As the content of the EVA polymer increased the increase of the crystal structure was obtained, visible through the increment of melting enthalpy. After ageing, temperatures associated with the fusion of the EVA PmBs shifted toward lower values. Also, the melting enthalpy was reduced, indicating the distortion of the crystal structure and degradation of EVA PmBs after thermo-oxidative ageing [8-10].

Table 1 DSC parameters of EVA PmBs before and after RTFOT

Samples	Conditions	Temperature Fusion Range [°C]	ΔH_m [J/g]	T_m [°C]
EVA-3	Unaged	40.4-82.8	1.78	64.9
	RTFOT	41.3-82.0	1.05	64.5
EVA-4	Unaged	39.7-82.1	2.68	65.9
	RTFOT	42.3-80.9	2.32	65.4
EVA-5	Unaged	34.9-89.5	5.21	66.8
	RTFOT	35.3-90.2	4.98	65.2
EVA-7	Unaged	30.5-87.6	7.24	67.6
	RTFOT	31.9-79.8	5.46	59.8

The frequency dependence of the rheological viscoelastic parameters, master curves of complex modulus, G*, complex viscosity, η^*, and phase angle, δ, for BIT and EVA PmBs before and after thermo-oxidative ageing are presented in Figure 1 to 3. The figures indicate that improved viscoelastic properties of EVA PmB can be observed over a wide frequency range. The curves of G*/f and η*/f show different behavioral trends at high and low frequencies. All EVA PmBs show a similar shifting of the complex modulus master curves towards higher values as the polymer content increases, particularly at low frequencies. At low frequencies, the differences in G*/f and η*/f curves are more evident in all EVA PmBs, which means the EVA modification has a stronger influence at lower frequencies. The complex viscosity master curves continuously decrease with the increase in frequency. The curve of η*/f develops higher values at lower frequencies, but it remains similar to that of unmodified BIT from intermediate to high frequencies. EVA PmBs shows an apparent increase in complex viscosity with increasing polymer content (Figure 2). It is evident that the η* BIT value approaches a constant value of 10^4 Pas at low frequencies. The BIT shows viscous behavior, the phase angle approaches 90°C at low frequencies (Figure 3) [7-10]. The values of phase angle master curves decrease with EVA modification. We can see evidence of a breakdown of time-temperature equivalence from the master curves of phase angle for EVA PmBs, particularly at intermediate and low frequencies and with a high content of added EVA (Figure 3). The breakdown of the smoothness of the master curve can be attributed to the structural changes of EVA with temperature which is evident from the DSC measurements (Table 1) [10]. The crystalline portions (packed polyethylene segments) of the EVA polymers may melt and networks weaken at a temperature of about 50°C [6, 9, 10]. The discontinuous "waves" on the phase angle master curve are more evident for highly modified, semi-crystalline EVA PmBs. The PmB with a 3 wt. % EVA polymer shows good master curves with small "branching" at temperatures of 50°C and 60°C, which is related to the start of semi- crystalline EVA polymer melting, as confirmed by the DSC measurement (Table 1). Despite the complex picture of phase angle master curve of EVA PmBs, the modification contributes to the decrease of the phase angle and the presence of a slight phase angle plateau. Lower δ and plateau formation mean that PmBs have better elastic behavior [7, 8, 10]. The changes in rheological master curves of EVA PmBs are noted after ageing under RTFOT conditions. The values of G* and η* increase after ageing, which is more evident at very low frequencies (Figure 1 and 2). This is related to the higher stiffness, which is a consequence of the oxidation process of BIT which is evident from the microphotography [12]. As the content of EVA increases the influence of ageing is reduced. After ageing the values of δ decrease, indicating lower elasticity (Figure 3) [3, 7, 9]. Discontinuous "waves" and "branching" on the phase angle master curves are reduced after ageing. This is more evident as the content of the EVA polymer increases. This is related to the destruction of the crystal structure of EVA PmB, as can be seen from the results of DSC (Table 1) and from the microphotography [12]. The crystal structure is disrupted and leads to a phase angle towards higher values, indicating the viscous behavior after ageing (Figure 3).

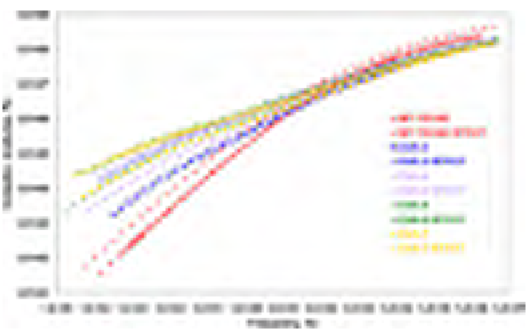

Figure 1 Complex modulus master curves for BIT and EVA PmBs

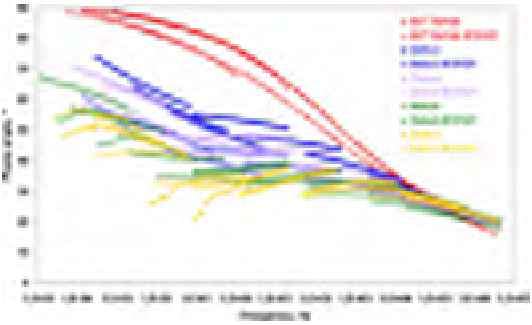

Figure 2 Complex viscosity master curves for BIT and EVA PmBs

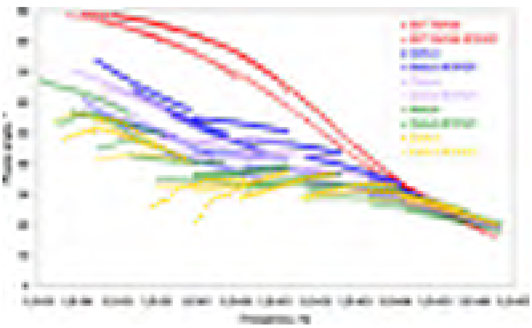

Figure 3 Phase angle master curves for BIT and EVA PmBs

Better rheological properties of EVA PmBs and better resistance to temperature can also be noted from the conventional tests results. The EVA addition in BIT increases the softening point value and the elastic behavior of the bitumen without a significant decrease of the penetration value. The best compromise between softening point, elastic recovery and penetration value, before and after RTFOT is reached by the EVA PmBs modified with 5 wt. % EVA polymer. These changes are in agreement with the changes in rheological properties of EVA PmBs.

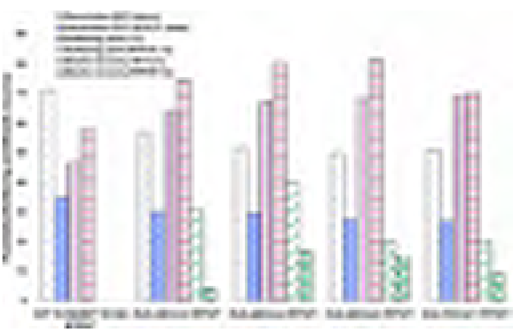

Figure 4 The relationships between the softening point, penetration and elastic recovery for BIT and EVA PmBs; before and after RTFOT

Critical SHRP temperature, T_C/°C, values for permanent deformation (rutting) are presented in Table 2. The critical temperatures of EVA PmBs are higher than that of the base bitumen. EVA PmBs with higher polymer contents have higher critical temperatures. This means better temperature resistance to permanent deformation, i.e. rutting [7, 8, 13].

Table 2 The critical SHRP temperature

SHRP	Samples				
	BIT 70/100	EVA-3	EVA-4	EVA-5	EVA-7
$T_{C,before\ RTFOT}$/°C where G*/sin≥1 kPa	65.1	86.0	93.5	91.1	85.9
$T_{C,after\ RTFOT}$/°C where G*/sin≥2.2 kPa	64.4	91.9	95.9	92.1	89.4
T_C/°C	64.0	82.0	88.0	88.0	82.0

5 Conclusions

The rheological properties of road bitumen are improved with EVA polymer modification as proven by the rheological master curves of G*, h*, d and by resistance to permanent deformation. EVA PmBs have better temperature resistance to permanent deformation under traffic frequencies than BIT which means better properties when used in road construction at high temperatures. The phase angle master curves have “branching” and “waves” due to the structural changes of EVA with temperature. After ageing the penetration values and the elasticity of EVA PmBs are decreased, while the softening point temperature is increased. This indicates that thermo-oxidative degradation is present. The rheological changes of EVA PmBs that occur after ageing are linked to a chemical change of the copolymer due to fusion of the crystallites. This leads to the reduction of discontinuous “branching” and “waves” on the phase angle master curve and an increase in viscous behavior after ageing.

References

[1] Carera, V., García-Morales, M., Gallegos, C. & Pérez-Lepe, A.: Bitumen chemical foaming for asphalt paving applications, Industrial & Engineering Chemistry Research, 49, pp. 8538-8543, 2010.

[2] Adedeji, A., Grunfelder, T., Bates, F. S., Macosko, C. W., Stroup-Gardiner, M. & Newcomb, D. E.: Asphalt modified by SBS triblock copolymer: structures and properties, Polymer Engineering and Science, 36, pp. 1707-1723, 1996.

[3] Carreau, P. J., Bousmina, M. & Bonniot F.: The viscoelastic properties of polymer modified asphalts, The Canadian Journal of Chemical Engineering, 78, pp. 495-502, 2000.

[4] Navarro, F. J., Partal, P., Martinez-Boza, F. J., Valencia, C. & Gallegos, C.: Rheological characteristics of groundtyre rubber-modified bitumens, Chemical Engineering Journal, 89, pp. 53-61, 2002.

[5] Kalantar, Z. N, Karim, M. R. & Mahrez, A.: A review of using waste and virgin polymer in pavement, Construction and Building Materials, 33, pp. 55-62, 2012.

[6] Polacco, G., Stastna, J., Vlachovicova, Z., Biondi, D. & Zanzotto, L.:Temporary Networks in Polymer-Modified Asphalts, Polymer Engineering and Science, 44, pp. 2185-2193, 2004.

[7] Bulatović, O. V., Rek, V., Marković, J. K.: Polymer Modified Bitumen, Materials Research Innovations, 16, pp. 1-6, 2012.

[8] Bulatović, V. O., Rek, V. & Marković, K. J.: Effect of polymer modifiers on the bitumen properties, Journal of Elastomers and Plastic, DOI: 10.1177/0095244312469964, 2013.

[9] McNally, T.: Polymer modified bitumen, Woodhead, UK, 2011.

[10] Airey, G. D.: Rheological characteristics of Polymer Modified and Aged Bitumens, Doctoral Thesis, Nottingham, 1997.

[11] Mezger, T. G.: The Rheology Handbook, Vincentz Network, Hannover, Germany, 2006.

[12] Bulatović, V. O., Rek, V. & Marković, K. J.: Rheological properties and stability of ethylene vinyl acetate polymer-modified bitumen, Polymer Engineering and Science, DOI: 10.1002/pen.23462, 2013.

[13] Nejad, F. M., Aghajani, P., Modarres, A. & Firoozifar, H.: Investigating the properties of crumb rubber modified using classic and SHRP testing method, Construction and Building Materials, 26, pp. 481-489, 2012.

ASSESSMENT OF AN APPROPRIATE MODIFIER CONTENT IN MODIFIED BITUMEN BASED ON THE MULTIPLE STRESS CREEP RECOVERY TEST

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Abstract

Construction of a durable bituminous pavements requires high quality materials such as bituminous binders with extended viscoelastic properties. The main objective of this paper was to compare and analyse high temperature viscoelastic properties of polymer modified bitumen (PMB) in order to assess appropriate modifier content. In this paper, there is described multiple stress creep recovery (MSCR) procedure in accordance to Superpave PG specification. Using the MSCR procedure, comparison with a traditional PG criteria was discussed. PG criteria and MSCR procedure were used to asses properties of polymer modified bitumens produced in Europe and to classified them in accordance to European standards. Additionally, new MSCR procedure was used to verify properties of binders used for construction of high traffic load pavements. Based on a conventional tests (penetration, softening point, elastic recovery and Fraass breaking point) as well as recovery and non-recoverable creep compliance obtained from MSCR test, amount of an appropriate modifier content was analysed. It was concluded, that although bituminous binders complies with European specification requirements, they are significantly different in terms of their rheological properties due to the modifier content. It was found, that polymer modified bitumen within the same hardness group exhibit different high temperature properties. This work confirmed poor correlation between elastic recovery in ductility test versus recovery from MSCR test.

Keywords: polymer modified bitumen (PMB), multiple stress creep recovery (MSCR), modification

1 Introduction

Superpave specification was developed by the Strategic Highway Research Program (SHRP) in the United States about 20 years ago. Research phase was launched in 1987, lasted 5 years and costs about 150 million dollars. Implementation stage of the program began in 1993 and up to now the requirements of Superpave are subjected to verification and changes [1]. The Superpave specification is using climatic zones to determine the performance grade of asphalt binders. Binders should comply the requirements for specific climate zone and at the same time, Superpave specification does not impose requirements on the composition and origin of the binders. Manufacturers of asphalt binders in the United States in order to increase the functional scope of the use of binders commonly modifies asphalt binders.
Superpave requirements in the original assumption were developed taking into account the average traffic speed and load. For heavy and/or slow traffic Superpave recommends increasing upper PG (performance grade) to the next level. As the result of this recommendation, manufacturers stiffer the binder by high polymer level modification. It hence increases the costs and raise the technological temperature of binder application.

The second major issue arriving when increasing PG is the lack of "sensibility of the test method" on the type of modification. In the United States for the purpose of stiffening binders and raise the type of PG, different types of modifiers (not polymer type) were started to be used. It has led to meet Superpave requirements, but it not improved pavement properties. The value of the parameter $G^*/\sin\delta$ is overwhelmingly dependent on the stiffness modulus value and in much lesser extent depends on the value of the phase angle. While the properties of bitumen modified with elastomers (e.g. SBS type) advantages over traditional road bitumens in an increase of the elasticity (that is not dependent on the stiffness of the binders), it is strongly dependent on the value of phase angle [2].
In response to the verification of the requirements of Superpave to adapt them to changing material modification trends and with take into account the nature of the traffic, the new Superpave requirements introduced the test method of multiple stress creep recovery (MSCR) [3].

2 Materials and testing methods

2.1 Materials

Binder samples used in this study were obtained from asphalt plants located in north-east, central, south and south-west parts of Poland. Two types of polymer modified bitumens PmB 25/55-60 (five samples) and PmB 45/80-55 (five samples) from two binder producers, were acquired. Each sample was obtained according to EN 58 (Bitumen and Bituminous Binders – Sampling Bituminous Binders) specification during binder unloading when transported to the asphalt mixture plant. Modified binders produced in Poland are obtained from Russian crude oil and modified with SBS polymer. It has to be noted, that according to the European requirements, bituminous producers providing PmB does not have to characterize content of the polymer used for modification (typical value of the SBS content in Polish market is about 3-5%).

2.2 Conventional testing methods

All binders in this study were tested with conventional classification test such as penetration at 25°C according to EN 1426, softening point according to EN 1427 and elastic recovery according to EN 13398. Additionally, samples were tested using dynamic shear rheometer according to EN 14770 and bending beam rheometer according to EN 14771 in order to analyze rheological properties and for the PG classification. Properties of polymer modified bitumen were obtained for non-aged as well as short-term (RTFO – Rolling Thin Film Oven) and long-term (PAV – Pressure Ageing Vessel) aged samples in accordance to Superpave classification. Properties of modified binders were evaluated according to EN 14023:2010 specifications (with Polish normative appendix NA). Test results obtained from BBR (bending beam rheometer) and DSR (dynamic share rheometer) apparatus were evaluated according to the US specifications (ASTM D6373 Standard Specification for Performance Graded Bituminous Binder).

2.3 Multiple stress creep recovery test method (MSCR)

Multiple stress creep recovery (MSCR) procedure allows to characterize high-temperature properties of bitumen and is used to designate the upper range of functional PG according to the AASHTO MP 19 [4]. This test shall be carried out in dynamic shear rheometer (DSR), using the measuring system parallel plates with a diameter of 25 mm and 1 mm gap in accordance to the procedure described in the AASHTO TP 70 [5]. The test procedure requires the repeated shear load per one second at constant stress applied to the bitumen sample after RTFO. Each load cycle is followed by a 9-second relaxation of material. Single cycle lasts 10 seconds and is repeated ten times on the 0.1 kPa and 3,2 kPa stress level. In each cycle of creep and relaxation, the strain change is recorded in the first and tenth of a second. Deformation analysis during the cycle can be separated for the elastic deformation (recoverable) and permanent deformation (non-recoverable). The result of the MSCR test is the percentage average elastic recovery R, calculated according to the formula (1) and (2) and non-recoverable creep compliance J_{nr} calculated according to the formula (3) and (4).

$$R = \frac{\sum_1^{10} \epsilon_r (\tau, N)}{10} \tag{1}$$

$$\varepsilon_r = (\varepsilon_1 - \varepsilon_{10}) \cdot 100 / \varepsilon_1 \tag{2}$$

$$J_{nr} = \frac{\sum_1^{10} J_{nr}(\tau, N)}{10} \tag{3}$$

$$J_{nr}(\tau, N) = \varepsilon_{10} / \tau \tag{4}$$

where:

τ stress;
N number of cycles;
ε_1 strain increment after the first second cycle (at the end of the phase of creep);
ε_{10} strain increment after the 10 second cycle (at the end of the relaxation phase).

The average percentage of elastic deformation R specifies the elastic properties of binders [6], and non-recoverable creep compliance J_{nr} specifies the permanent deformation sensitivity of the material. The increasing of J_{nr} value affects decrease resistance to permanent deformation [7, 8]. The requirements of AASHTO MP 19 [4] bring additional classification within a single performance grade. Standard sets four classes depending on traffic load (ESAL) includes: standard – S, high – H, very high – V and extremely high – E (see table 1) [6].

Table 1 The requirements of resistance to permanent deformation of asphalt binders according to AASHTO MP 19

Classes of traffic load				
Traffic characteristics	Standard S	High H	Very high V	Extremely high E
	‹70km/h ‹10 mln ESAL	20÷70 km/h 10÷30 mln ESAL	›20 km/h ›30 mln ESAL	‹20 km/h ›30 mln ESAL
Requirement (binder after RTFOT)	Jnr3,2 ≤ 4,0 [kPa^{-1}]	Jnr3,2 ≤ 2,0 [kPa^{-1}]	Jnr3,2 ≤ 1,0 [kPa^{-1}]	Jnr3,2 ≤ 0,5 [kPa^{-1}]

3 Tests results

During this study basic classification and advanced rheological test were conducted on different samples of modified binders to compare the repeatability of properties of binders from Polish market. Results of binder penetration tests conducted at 25°C, softening point temperature by R&B method, Fraass breaking point and elastic recovery are shown in Table 2 for 25/55-60 and 45/80-55 polymer modified bitumen.

Table 2 Basic properties of polymer modified bitumen

Sample of PMB binder	Penetration at 25°C [0,1mm]	Softening point temperature by R&B [°C]	Fraass breaking temperature [°C]	Recovery elastic [%]
25/55-60_1	33	64,8	-11	68
25/55-60_2	36	66,2	-16	82
25/55-60_3	25	69,6	-17	75
25/55-60_4	32	59,4	-10	81
25/55-60_5	41	60,8	-22	76
min value	25	59,4	-22	68
max value	41	69,6	-10	82
SD	5,47	3,72	4,35	5,30
45/80-55_1	46	58,0	-14	77
45/80-55_2	49	56,2	-11	79
45/80-55_3	53	62,0	-18	84
45/80-55_4	46	56,4	-11	62
45/80-55_5	56	62,0	-15	86
min value	46	56,2	-18	62
max value	56	62,0	-11	86
SD	3,67	2,62	2,64	8,63

It can be noticed, that for the same binder type (25/55-60 or 45/80-55) results of penetration and softening point are diversified but except one case for 25/55-60 sample, binders met requirements (table 3).

Table 3 Requirements of polymer modified bitumen in according to PN-EN 14023:2010 (Appendix NA)

PMB type	Requirements			
	Penetration at 25°C [0,1mm]	Softening point temperature by R&B [°C]	Fraass breaking temperature [°C]	Recovery elastic [%]
25/55-60	25/55	≥ 60	≤ -10	≥ 50
45/80-55	45/80	≥ 55	≤ -12	≥ 50

For analysed binders large diversification of test results was recognized in Fraass breaking point with a differences of up to 10°C and in elastic recovery over 20%. According to the Polish specifications (Appendix NA for EN 14023), for the polymer modified bitumen the required values are: 50% for elastic recovery and -10°C or -12°C for breaking point for PMB 25/55-60 and PMB 45/80-55 respectively. This fact shows significant differences between similar types of binders.

3.1 Assessment of the PG gradation

Based on the G*/sinδ value and Superpave requirements, upper limit of the performance grade temperature has been determined for polymer modified bitumen based on the original and RTFO aged binder. In accordance to Superpave requirements, the critical temperature of PMBs were determined using BBR apparatus based on the stiffness modulus and m parameter on PAV aged samples. PG gradation of tested polymer modified bitumen containing low and high temperature limit are shown in Figure 1.

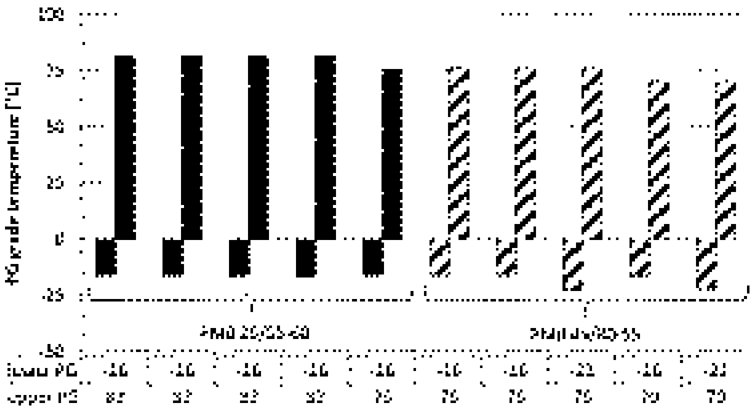

Figure 1 Performance gradation of tested polymer modified bitumen

Based on the PG gradation of tested binders it can be concluded, that all modified bitumens shown upper limit of the performance grade temperature above 70°C. Higher temperature (about 82°C) were obtained for PMB 25/55-60. High upper limit of the performance grade temperature, determined using DSR apparatus, indicate high rutting resistance.
Based on the conducted analysis of low BBR temperature, it was concluded that PMBs does not exhibit favorable low temperature properties, especially when compared with the typical critical winter pavement temperatures in Poland (from 28°C to -34°C).

3.2 Assessment of multiple stress creep recovery and elastic recovery

Large diversification of test results of polymer modified bitumens both in classification tests according to European Standards and obtained a significant stiffness of binder on the basis of assessments by PG grade, demonstrates difficulties in the proper evaluation of the polymer modified bitumen and, in particular, the contents of the polymer. The MSCR test method allows the assessment of appropriate level of polymer modification of bitumen based on the elastic recovery R with respect to the non-recoverable creep compliance J_{nr} of the binders and to the traffic load. Figures 2 and 3 shows test results for PMB 25/55-60 and PMB 45/80-55, respectively, depending on the elastic recovery and the J_{nr} parameter from MSCR test with 3,2 kPa stress level. A straight line with dots in figures presents the empirical curve for proper polymer content in bitumen. Results located over the curve indicate the correct amount of polymer.

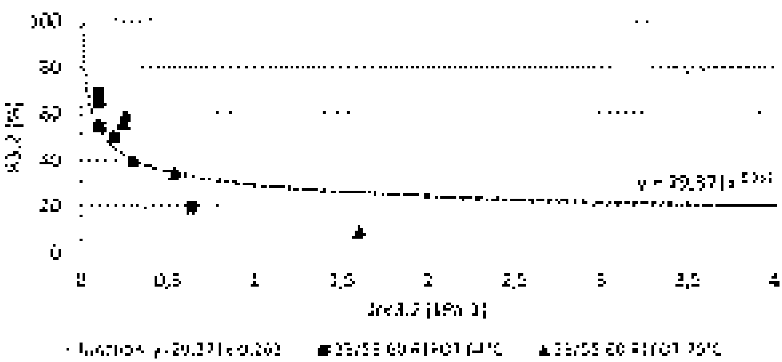

Figure 2 Dependence of elastic recovery R and the J_{nr} parameter from MSCR with 3,2 kPa stress level for PMB 25/55-60

On the basis of an analysis of the test results obtained from MSCR test, some of the modified binders are located below the curve. For PMB 45/80-55 sixty percent of tested binders are located below the reference curve, which proves that they contain insufficient amount of polymer. Analysis of the results of non-recoverable creep compliance J_{nr} shows higher stiffness of the binder from group PMB 25/55-60. For most of these binders non-recoverable creep compliance not exceeded 0.6 kPa^{-1} value. It can be noticed, that in spite of the insufficient level of polymer modification, most binders complies the requirements of the EN standards and there is possible to evaluate the PG grade according to Superpave (AASHTO M 320-09 Performance-Graded Asphalt Binder).

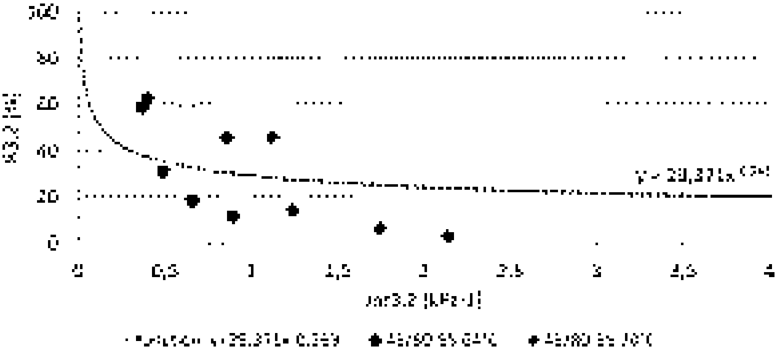

Figure 3 Relationship between elastic recovery R and the J_{nr} parameter from MSCR with 3,2 kPa stress level for PMB 45/80-55

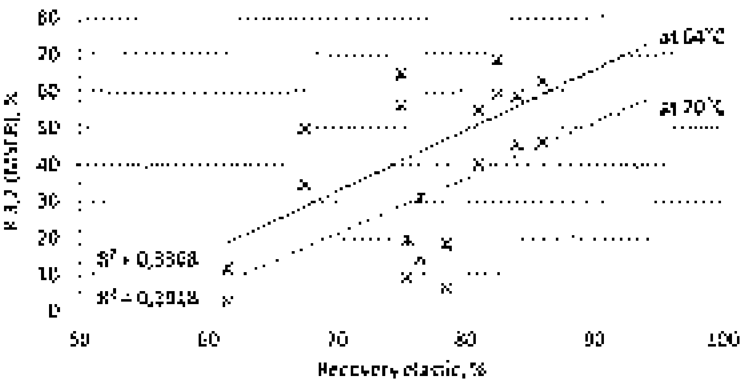

Figure 4 Relationship between elastic recovery from ductility and elastic recovery from MSCR

In order to compare methods used to evaluate elasticity of modified binder, in Figure 4 there is presented correlation between elastic recovery from ductility and elastic recovery from MSCR test. Based on the results presented in Figure 4, no correlation can be observed for both the 64°C and 70°C tests temperature ($R^2 < 0,35$).

4 Conclusions

Based on the tests conducted and analysis of the polymer modified bitumen produced in Poland, the following conclusions can be drawn:

1. The bituminous binders exhibit large diversification of test results, but, except few cases, meet all of the current European requirements.
2. Based on the stiffness modulus and m parameter obtained from BBR and on the G*/sinδ parameter from DSR tests, it can be noticed that all bituminous binders demonstrate very good performance in high temperature with limited resistance to low thermal cracking.
3. All of the tested modified binders exhibit over 50% value of the ductility elastic recovery, but only half of tested modified binders exhibit relevant polymer content in accordance to MSCR test method.
4. According to some reports [9,10] published in recent years about difficulties in the application of the polymer modified bitumen and not sufficient quality, it seems necessary to incorporate an additional assessment by MSCR methods. Multiple Stress Creep Recovery method allows for assessment of appropriate amount of elastomer used for modification and to eliminate binder with weak elastic properties.

References

[1] McDaniel, R.S., Leahy, R.B., Huber, G.A., Moulthrop, J.S., Ferragut, T.: The Superpave Mix Design System: Anatomy of a Research Program. NCHRP Final Report, document 186. 2011

[2] Asphalt Institute. Implementation of the multiple stress creep recovery test and specification. Asphalt Institute Guidance Document April 2010, dostępny na http://www.asphaltinstitute.org/Upload/MSCR_Implementation_Guidance_1058794596_542010142630.pdf

[3] Król, J., Matraszek, K.: Bitumen high temperature properties characterised by multiple stress creep recovery test (MSCR). Drogownictwo 10/2012, pp. 321-324.

[4] AASHTO MP 19-10 Standard Specification for Performance-Graded asphalt Binder Using Multiple Stress Creep Recovery (MSCR) Test

[5] AASHTO TP 70-09 Standard Method of Test for Multiple Stress Creep Recovery (MSCR) Test of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)

[6] D'Angelo, J.: New high-temperature binder specification using multistress creep and recovery. Development in Asphalt Binder Specifications. Transportation Research Circular E-C147. 2010.

[7] Dreessen, S., Planche, J.P., Gardel, V.: A new performance related test method for rutting prediction: MSCRT. Advanced Testing and Characterization of Bituminous Materials. Vol. 1/2009 pp. 971-980.

[8] Gajewski, M., Wróbel, A., Jemioło, S., Sybilski, D.: Effect of rheological properties of binders on HMA rutting. XIV International Conference Computer Systems Aided Science, Industry and Transport 2010.

[9] Radziszewski, P., Kowalski, K., Król, J., Sarnowski, M., Piłat, J.: Quality assessment of bituminous binders based on the viscoelastic properties: polish experience, Journal of Civil Engineering and Management, Vol. 20(1), 2014 pp. 111–120.

[10] Teugels, W.: Bitumen quality, performance and specifications. What can we learn and what are the limitations. XXVIII PSWNA Seminar 2013 – Warsaw, Poland.

EXPERIMENTAL STUDY ON THE ENHANCEMENT OF MECHANICAL PROPERTIES OF BITUMINOUS MASTICS AT HIGH STRAINS

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Abstract

The paper presents the results of a laboratory investigation aimed at evaluating the effects of filler on the rheological properties of bituminous mastics. More specifically, the mechanical behaviour of mixtures made with bitumen and inorganic aggregate is analyzed: selected fines from Municipal Solid Waste Incinerator (MSWI) ashes were used. The interaction between binder and aggregate determines a change in the rheological behaviour. Experimental data show a viscoelastic behaviour in the usual operating temperatures range and in a dynamic stress configuration characterized by high strains. It can also be observed that the thermal susceptibility of the mastic markedly depends on the content of filler, which is responsible for the variations of stiffness and resilience, measured in terms of complex modulus and phase angle. In order to examine the rheological effect of inorganic filler, tests were performed with the objective of checking the mastic behaviour at the edge of the linear viscoelastic region. Above the linear viscoelastic threshold, the bituminous mastic is significantly sensitive to the presence of filler and a statistically robust protocol is required. The experimental data were obtained by means of a dynamic shear rheometer (DSR) and synthesized with a viscoelastic model. For the interpretation of the results, master curves of the main performance indices were calculated, being related to the filler content. As expected, the results confirm that artificial aggregate from MSW incinerators can be used in order to improve mastic properties.

Keywords: bituminous mastics, inorganic aggregate, marginal material, rheology

1 Introduction

In recent years, marginal and industrial waste materials have been widely used in various civil and industrial engineering sectors. These materials have demonstrated a good capacity of integration with the traditional materials and an ability to substitute aggregates of natural origin, contributing to a saving of natural aggregates extracted from quarries, and therefore to environmental conservation. In particular, the diffusion of incinerators has made residues available from the combustion of municipal solid waste (MSW) in the form of slags, which are worthwhile studying for their use in road pavements [1-8]. The MSW slags, at one time disposed of in dumps, cannot be used as they are, because they often contain unsuitable elements (metal residues like nails, bolts, cutlery) and tend to release substances (heavy metals or very fine particles) that are potentially polluting for soils and groundwater. The material may also contain components incompatible with the bitumen under the electro-chemical profile, which impede the mixing or performances of the asphalt concretes. The slags are therefore utilizable only after a selection process. It should be remembered that the major strong point of this waste material is its low cost. For this reason, economic fractionation methods or selection processes (vitrification) of the aggregate should be used to provide a suitable aggregate for use in construction.

In this paper it was decided to use only the finest fraction (the filler, D < 0.075 mm) of the MSW slags to study the variation of the mechanical properties of the bitumen and the interaction with the filler. The hot mixing of these two elements, filler and bitumen, gives a semisolid mastic, which influences the performances of the asphalts produced. In particular, it was demonstrated that the presence of the filler in the bitumen improves the durability of the mix, mechanical resistance and pavement performances: an increase of the filler content increases the stiffness of the mastic; instead, a reduction of the bitumen content reduces the stability of the compound. Although there are some studies that demonstrate a reduction of the principal physical-mechanical characteristics of the mixtures containing marginal materials of industrial origin, compared to analogous mixes with conventional materials, recent studies have evidenced the improvements obtainable with the use of filler from MSW slags [5-8]; starting from these, based on the study of the mechanical behaviour of the mastics in the linear viscoelastic field (LVE), it was decided to extend the characterization to the limits of this region, i.e. to the conditions in which the material is subjected to high strain levels, when the normal models start to lose their efficacy. With this aim, a dynamic shear rheometer (DSR) was used to support the dynamic analysis (DMA), in a sinusoidal oscillatory regime, of these materials and to determine the principal rheological properties (complex modulus, phase angle) of the bitumen. The use of a DSR for the characterization of mastics has long been debated, because, in the past, the material drove the instrument to its operative and mechanical limits. In this research an instrument was used that could support the necessary regimes of force and strain [9-13, 15]. Moreover, the quantity of filler added to the bitumen, the dimensions of the filler (maximum nominal diameter) and the states of torsion reached are compatible with the current technical standard and are well within the working limits of the instrument. The results of the analyses conducted during this study have identified the effects of the rate of filler on the mechanical behaviour and consistency of the mastic.

2 Experimental programme and research setup

The aim of this study is to evaluate the improvement in the mechanical properties of bitumen with the addition of filler, seeking the optimal content of the latter, which allows the performances to be optimized at the limit of the LVE field. The performances of the bitumen and mastic were analyzed using a dynamic shear rheometer (DSR), in particularly delicate conditions that are intensified with the increase in the rate of filler. The empirical characteristics of the binders and mastics were analyzed first in order to classify and select them. In the next phase, after having determined the limits of the LVE field, laboratory tests for the rheological characterization of the bitumen and mastics were conducted in controlled conditions, varying the rate of filler.

2.1 Materials

Different types of traditional bitumen were used, with variable origin (all from Italy) and performances. The bitumen specimens were prepared and stored according to the European standard EN 58. The binders were chosen on the basis of their physical-mechanical characteristics: modified bitumens were excluded and those not in line with the prescriptions of the current European standards. The slags from MSW combustion, collected from a North-East Italy plant, were heated in an oven at a temperature of 105 °C to eliminate the residual moisture. Samples were then sieved and the fraction passing through a 0.075 mm diameter sieve was collected. The mastics prepared for the tests contained the following quantities of filler: 25%, 50%, 75% and 100% of the mastic weight. These values are in line with the typical prescriptions for asphalt mixes for road use, and are consistent with the traditional types and with research conducted in this field [14]. In order to guarantee that the standards defined by the project protocol were met, all the preparation and mixing phases were monitored using an infrared NEC Thermo Tracer TS9260 camera (T-IR). The test were entirely performed at the Road Laboratory of the University of Padua.

2.2 Empirical Analysis

The following tests were performed: penetration at 25 °C (EN 1426), softening point (R&B) temperature (EN 1427), Fraass breaking point (EN 12593), dynamic viscosity at 135 °C (EN 13702), mass-loss on heating – LOH (EN 12607/1), penetration at 25 °C after LOH (EN 1426). The initial empirical analysis identified three classes of penetration: 35/50, 50/70 and 70/100 mm/10; a summary of the first two tests for the bitumens and relative mastics is presented.

2.3 Dynamic and Rheological Analysis

The tests, conducted with an AntonPaar MCR 302 DSR, were done in sinusoidal oscillatory regime, varying the frequency (f) and temperature. Two geometries are usually used in this type of test, which have a diameter of 8 mm and 25 mm. However, the preliminary tests demonstrated a discrepancy between the results obtained in the two cases. For this reason it was decided to use the two testing plate geometries (Table 1), again in accordance with the prescriptions in the technical standard. The gap depends on the geometry and amount of material tested (Table 1). The tests were repeated 3 times for each type of material and the results were then averaged. The tests (in frequency sweep) were conducted in accordance with the prescriptions indicated in the standard EN 14770: the bitumen, or mastic, was poured onto the lower plate of the DSR at a temperature of 130 °C (±10°C). The value of normal force was fixed: during the descent of the upper plate (‹ 0.5 N), to avoid the mastic suffering a pre-stress due to excess material; during the trimming phase; during the test (N_f= 0 N), to maintain the contact between plate and bitumen; during the conditioning phases. The limit of the linear viscoelastic field was determined for all the materials with separate tests, in amplitude sweep: strain limit was determined on the average of five specimens and for all temperatures (e.g. 1% to 5%).

Table 1 Test conditions and spindle geometry

Testing geometry	Diameter [mm]	Gap width [mm]	Temperature [°C]	Step Temp. ΔT [°C]	Reduced Frequency [Hz]
PP08	8	2	10 to 35	5	0.016 to 16
PP20	20	1	30 to 80	5	0.015 to 16

For the construction of the isothermal curves of the complex modulus (G*) and the phase angle (δ), different tests were done, in frequency sweep, with gradual 5 °C temperature increases (ΔT), on three identical specimens. The spindle compliance condition was verified directly on the curves in frequency sweep, conducting the tests with both the geometries at temperatures of 30 and 35 °C. The reproduction of the master curves of the complex modulus |G*| and phase angle δ is at the arbitrary reference temperature of 25 °C. The single isothermal curves represent the response of the material to a loading cycle in frequency sweep. These were shifted utilizing a polynomial factor present in the literature [14]. In this connection the principle of time-temperature superposition (TTS) is considered valid and the materials are considered "thermo-rheologically simple": an increase in the temperature reduces the resting times of the structural processes associated to the strain and flow. If this reduction is the same for all the times of the resting spectrum, an increase of the temperature determines in the diagram log(|G*|)-log(f) a horizontal shift of the response G(f) towards shorter times and without substantial vertical movements [15]. All the materials are represented with the respective average model curve, between 1.0E-05 and 1.0E+02 Hz; the lines relative to the extremities of the curve differ according to the maximum (min torque) and minimum temperature (max torque) reachable with the tests in frequency sweep.

3 Results and discussion

The master curves presented in this paper were calculated with reference to a model proposed in previous studies [11, 12, 14, 15] and suitable to represent these materials. The analytical formulation, although not simple, is suited to the programming of iterative cycles that allow a well-defined curve to be reached with five parameters (G_g, R, k, w, δ_∞). For the modelling, as suggested by the authors, it was initially decided to set the maximum value of the complex modulus at 1 GPa. Nevertheless, with the analysis procedure and available data, it was observed that the model adhered better to the experimental data if the first constraint was eliminated. Although the optimization of the model is at an initial stage, for the presentation of the results of this study it was decided to add another degree of freedom, also freeing the constraint of the minimum value of the complex modulus ($Gf_{\rightarrow 0} \neq 0$).

3.1 Unaged unmodified bitumen pen 35/50

Between types available, for this class of penetration, two bitumens were evaluated as being suitable: 35/50Li and 35/50TO. The empirical profile shows that the mastic is more consistent and less sensitive to the temperature of the original bitumen (Table 2). Although the bitumen is the originator of the mechanical behaviour of the mastic, the influence of the filler content is clear on the two bitumens and shows in a different way. The values of the complex modulus are comparable on average and increase with the filler content, the bitumen 35/50TO acquires greater stiffness with the same added filler. The phase angle and, therefore, the resilience of the mastic do not present similarities: in the first case (35/50Li) the resilience tends to diminish with the addition of filler; in the second case (35/50TO) the comparison with the original bitumen depends on the test frequency (figures are omitted due to lack of space).

Table 2 Bit. 35/50. Softening point temperature [°C] and penetration [mm/10]

Property/mastic	0% Filler		25% Filler		50% Filler		75% Filler		100% Filler	
Sign	Li	TO	Li	TO	Li	TO	Li	TO	Li	TO
Soft. Point [°C]	47	48	50	49	52	52	58	57	64	65
Pen. [0.1 mm]	37	39	39	34	30	28	25	23	22	23

3.2 Unaged unmodified bitumen pen 50/70

Three bitumens were examined (50/70Ve, 50/70Li, 50/70TO) that respected the indications in the standard. The expectations about the effect of the filler content were confirmed, for all the empirical tests. Although the bitumens belong to the same class, the filler produces a different evolution of the values of penetration and softening temperature (Table 3), more marked with respect to the class 35/50 pen.

Table 3 Bit. 50/70. Softening point temperature [°C] and penetration [mm/10]

Mastic Property	0% Filler			25% Filler			50% Filler			75% Filler			100% Filler		
Sign	Ve	Li	TO	Ve	Li	TO	Ve	Li	TO	Ve	Li	TO	Ve	Li	TO
Soft. [°C]	53	45	49	54	47	50	54	49	52	60	54	57	65	60	64
Pen. [0.1 mm]	68	65	54	57	53	48	42	43	36	32	37	34	24	29	27

In all three cases examined an increase of the stiffness is observed, in agreement with the increase in filler content, while the resilience of the mastic reduces. The cause-effect ratio between bitumen and filler is not comparable. In the first case (50/70Ve) the filler determines

a shift of the stiffness towards higher levels of G*; the phase angle remains relatively low ($\delta < 80°$) and it is not possible to predict the position of the lower horizontal asymptote: as the temperature rises the mastic loses its resilience, shown by the original bitumen, therefore the value of δ_∞ cannot be assumed constant and equal to 0°.
The results obtained from the tests on the bitumens 50/70Li and 50/70TO, and relative mastics, are congruent with the expectations as regards the stiffness (G*). Nevertheless, the relationship between loss of resilience and filler is strongly influenced by the original bitumen: the 50/70Li appears to be less sensitive than the others; while the 50/70TO appears to be unaffected by the filler at low rates (25% and 50%) to then clearly lose resilience with higher rates (75% and 100%). In this phase tests were not done at temperatures lower than 10 °C, so it is not possible to make precise predictions on the position of the value of the glass modulus. The comparison between the moduli of the three bitumens shows that the 50/70Ve evolves with a lower gradient and towards a lower glass asymptote.

3.3 Unaged unmodified bitumen pen 70/100

The selection of the bitumens produced three samples: 70/100Ve, 70/100Li and 70/100TO. It is not possible to identify a connection between filler content and sensitivity of the bitumen to the variation of the temperature. The ratio is not linear and reflects the dominance of the filler (e.g. 75-100%). Although there are slight differences (see Table 4, penetration at 25% and softening point at 75% filler content), empirically the three bitumens are assimilable. This analogy is completely lost observing the results of the tests with the DSR: both the stiffness and resilience have different evolutions with respect to the loading frequency. The bitumen 70/100Ve is the most sensitive to the filler content: it was originally, under the same conditions, the least rigid and most resilient. The derived mastics show a clear reduction of the resilience, to the point that it is reasonable to assume a lower horizontal asymptote at around 45°.

Table 4 Bit. 70/100. Softening point temperature [°C] and penetration [mm/10]

Mastic Property	0% Filler			25% Filler			50% Filler			75% Filler			100% Filler		
Sign	Ve	Li	TO	Ve	Li	TO	Ve	Li	TO	Ve	Li	TO	Ve	Li	TO
Soft. [°C]	46	44	46	48	47	47	50	49	49	56	52	52	61	60	60
Pen. [0.1 mm]	72	75	75	62	67	60	42	49	49	41	40	40	34	31	32

Table 5 Stiffening and melting effects of filler content

[illegible]

An important result obtained from the tests regards the operative limits of the mastic produced with this filler: the greatest effects on the phase angle are located beyond the frequency of 1 Hz and become increasingly more marked. Table 5 shows that all the bitumens only undergo an effective increase of the performances if the rate is above 50% in weight; however, in all the cases, the increased stiffness is limited within same order of magnitude. While the bitumen 35/50TO shows anomalous behaviour at low rates, in general all the other binders establish an effective bond with the filler with stiffness increments that increase with the frequency.

4 Conclusions

A campaign of tests has been formulated for the characterization of unmodified and unaged bitumens that has confirmed the weakness of the empirical tests in the comparison with the results of the DSR, in particular as regards the resilience of the mastic.
As expected, the bitumen tends to show Newtonian type behaviour at high temperatures ($G_0 \rightarrow 0$, T_{max}=80 °C), unlike the mastic. In this particular region of the viscoelastic behaviour, the mastic supplies performances that initially depend on the original bitumen. As the rate of filler increases, this tends to compensate for limits of the initial bitumen, taking stiffness and resilience to comparable levels, unconnected with the penetration class of the original bitumen. At low temperatures, the complex modulus seems to be far removed from the threshold of 1 GPa, for all penetration classes. This classification does not reflect the physical-chemical interaction between the two elements following the mixing, and it is not simple to predict the effects on the loss of resilience. The complex modulus increases significantly with the filler content, therefore this material supplies stiffness to the original bitumen. On the other hand, the resilience is lowered, especially at the extreme limits of the frequencies and temperatures. The greatest benefits are therefore found in the interval of central frequencies, between 0.001 and 1 Hz, where the phase angle results as practically independent of the filler. The domination of the filler, as expected, appears at the extremes of the investigated frequency interval, especially the upper one, i.e. where the instrumental limits of the test configuration are reached. The influence of the stiffness of the material on the results of the test with the DSR should be taken into account: it is reasonable to suppose that the instrument and test configuration play an important role, that there will be errors that increase in intensity with the increase of the filler content, complexity of the mastic and approaching the operative limits of the instrument (max and min torque). The tested mastics are classified as thermo-rheologically complex materials, so it is reasonable to hypothesize that ageing will produce more marked effects on the filler bitumen interaction. The model used to construct the master curves allows a simple and efficient calculation, but tends to fail for mastics with high percentages of filler. It seems that there is a strong physical-chemical interaction between the two materials: the aggregate dominates the bitumen if dosed at above 50% and can no longer be considered as a solid dispersed phase.

Statement

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References

[1] Garrick, N.W. & Chan, K.L.: Evaluation of domestic incinerator ash for use as aggregate in asphalt concrete, TRRecord, 1418, pp. 30-34, 1993.

[2] Zhang, X., Gress, D., Karpinski, S. & Eighmy T.T.: Utilization of municipal solid waste combustion bottom ash as a paving material, TRRecord, 1652, pp. 257-263, 1999.

[3] Ogunro, V.O., Inyang, H.I., Hooper, F., Young, D. & Oturkar, A.: Gradation control of bottom ash aggregate in Superpave bituminous mixtures, Journal of Materials in Civil Engineering, 16(6), pp. 604-613, 2004.

[4] Flyhammar, P. & Bendz, D.: Leaching of different elements from sub-base layers of alternative aggregates in pavement constructions, Haz. Mat. Journal, 137, 2006.

[5] Huang, C.M., Chiu, C.T., Li, K.C. & Yang W.A.: Physical and environmental properties of asphalt mixtures containing incinerator bottom ash, Haz. Mat. Journal, 137, 2006.

[6] Xue, Y., Hou, H., Zhu, S. & Zha J.: Utilization of municipal solid waste incineration ash in stone mastic asphalt mixtures: Pavement performance and environmental impact, Construction and Building Materials, 23, pp. 989-996, 2009.

[7] Dabo, D., Badreddine, R., De Windt, L. & Drouadaine, I.: Ten-year chemical evolution of leachate and municipal solid waste incineration bottom ash used in a test road site, Haz. Mat. Journal, 172, pp. 904-913, 2009.

[8] Pasetto, M. & Baldo, N.: Mechanical characterization of bituminous mixtures with bottom ash from Municipal Solid Waste Incineration (MSWI), Proceedings WasteEng 10, Beijing (RPC), CD-ROM, pp. 6, 2010.

[9] Anderson, D.A., Christensen, D.W. & Bahia, H.: Physical Properties of Asphalt Cement and the Development of Performance-Related Specifications, Journal of the Association of Asphalt Paving Technologists, 60, pp. 437–532, 1991.

[10] Airey, G.D. & Rahimzadeh, B.: Combined bituminous binder and mixture linear rheological properties, Construction and Building Materials, 18, pp. 535–548, 2004.

[11] Md. Yusoff, N.I., Shaw, M.T. & Airey G.D.: Modelling the Linear Viscoelastic Rheological Properties of Bituminous Binders, Construction and Building Materials, 25, pp. 2171–2189, 2011.

[12] Marasteanu, O. & Anderson, D.A.: Improved Model for Bitumen Rheological Characterization, Proceedings Eurobitume, Luxembourg, paper no. 133, 1999a.

[13] Mezger, T.G.: Rheology Handbook, 2nd Edition, William Andrew Publ., 2006.

[14] Md. Yusoff, N.I.: Modelling the Linear Viscoelastic Rheological Properties of Bituminous Binders, PhD Thesis submitted to the University of Nottingham, 2012.

[15] Chailleux, E., Ramond, G. & De La Roche, C.A.: Mathematical-based Master Curve Construction Method Applied to Complex Modulus of Bituminous Materials, Journal of Road Materials and Pavement Design, 7, pp. 75–92, 2006.

EFFECT OF BITUMEN ORIGIN ON BEHAVIOR OF COLD RECYCLED MIXES USING FOAMED BITUMEN TECHNIQUE

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Abstract

Within the European research project COREPASOL supported by a group of countries associated in the Conference of European Directors of Roads, the effect of distilled bitumen gained from different locations on the characteristics (indirect tensile strength, water susceptibility and stiffness) of cold recycled asphalt mixtures has been assessed as a partial task. In the first step, the optimum foaming water was evaluated for each bituminous sample by optimizing the relationship between the expansion rate and the half-time. For the optimized foaming water content, one cold recycled mix design with different foamed bituminous binders has been used. The behavior of all experimental mixes has been tested by indirect tensile strength test done after 7 and 14 days for cured specimens, whereas the curing procedure respected Czech technical specification TP208. In parallel, stiffness testing has been done for each mix variant and a group of specimens has been water soaked between 7th and 14th day. These specimens were used for water susceptibility testing. After the first set of tests has been done, two foaming agents were used for selected bitumen represented by additives provided by Iterchimica and MeadWestVaco. For bitumen doped by such agents, the foam characteristics were evaluated again with a distinctive improvement in both foam characteristics particularly for one of the agents. Also this foamed bitumen has been later used for cold recycled mix laboratory production and testing. This paper presents the results obtained and findings with recommendations.

Keywords: cold recycling mixes, foamed bitumen, foaming agent, indirect tensile strength, stiffness modulus, water susceptibility

1 Introduction

Foamed bitumen lends itself to use as a binder for stabilizing various grain materials, including asphalt materials from the original, reclaimed asphalt pavement (RAP). Energy savings, sustainable road construction development and conservative use of material particularly in relation to the bituminous binder are the most important issues in the road building industry. The aspects represent a very strong incentive to use foamed bitumen technology all over the world, both from the economic and the environmental aspects [1]. Foamed bitumen technology can no longer be considered a real innovation. The technology of controlled foamed bitumen production was patented as early as in 1956 by Professor L. H. Csaniy of the Engineering Experiment Station of Iowa State University, originally as an improved binder for soil stabilisation in the construction of American military roads on the Pacific islands using solely volcanic ashes. After several years, the technology was abandoned; subsequently it was bought by Mobil Oil Australia that designed a fundamental improvement with which the technology has survived to this day. The mix spread significantly primarily in Australia already in the 1970's as a consequence of the oil crisis and skyrocketing prices of oil products which

resulted in a search for alternative and economically viable technologies for road building and reconstruction. Until 1993, the method was covered by a patent and any interested parties could only buy the licence for the technological method from Mobil Oil. When the patent on the technological method expired the technology for bitumen foam production rapidly spread in a number of countries. From the perspective of technical development of foamed bitumen technology application, particularly in the field of cold recycling, this expansion was most supported by company Wirtgen. Besides, alternatives which can be classified today as warm asphalt mixes with applicability in traditional mixing plants (applications known from Norway, Great Britain or USA) have also found their use. With respect to the fact that the technology has gradually found its way into pavement reconstruction in a number of industrial countries, the myth according to which any technology using foamed bitumen can only be used to build pavements solely in developing countries has finally been destroyed.
At present, there is a broad range of design methods for foamed bitumen mixes. Each of them differs in the mineral component grain size, filler type and content, water content in the mix, as well as the purely experimental issues related to the test specimen sizes, type of mixing, test specimen compaction and curing. The design procedures for foamed bitumen mixes also strongly depend on the type and quality of the reclaimed aggregate since different countries have different pavement compositions from the point of view of the mix contents traditionally used [2] and the structure components themselves.
In general, any bituminous binder which comes into contact with water when hot starts foaming and increases its volume several fold. However, this is not desired in most cases. The principle of foamed bitumen production is pumping air (under the pressure of approx. 10 bar) and water (1-5 %-wt. under the temperature of 15-25°C) under the pressure of 4-5 bar into the hot bitumen binder (50/70 – 160/220 applying temperature of 170-190°C). In contact with the hot bituminous binder, water immediately changes to steam. This is gradually captured and closed in minute bitumen foam bubbles; once it has transformed into steam the foam expands and increases in volume several fold. In this state, it is dosed into the aggregate. The entire foaming process takes place in an expansion chamber where the hot bituminous binder is fed and the steam and air added through jets. Subsequently, the resulting mix is pressure-pumped into the aggregate. When the foaming is maximal the bituminous binder bubbles gradually collapse; their surface attempts to adhere closely primarily to the fine aggregate particles which generates bitumen binder droplets that subsequently join one another and create a thin bitumen film; this coats the larger aggregate particles. Here, it is necessary to mention the effect of thermal susceptibility of the bituminous binder, or sudden changes of bitumen viscosity in contact with cold water during the foaming process which affects the foaming capacity and quality of the foam produced that must be taken into consideration. Thermal susceptibility can be defined as a change of bitumen viscosity depending on temperature changes [3]. In comparison to a bituminous binder or emulsion, foamed bitumen has much greater surface and different surface tension which allows sufficient coating of particularly the fine aggregate particles with a smaller quantity of bitumen. In this regard, the content of fine particles in the mix design is important as the mix operates on the principle of coating the particles with the foam and the created mastic thus created subsequently binds the larger aggregate grains together. A part of the foam is naturally caught on the surface of larger aggregate grains.
In general, the foamed bitumen quality is expressed as the proportion of the maximum bitumen foam volume achieved and the volume of the original bitumen binder when foamed, that is, the expansion ratio (ER). With superior quality foamed bitumen, the expansion ratio should be at least 8-15. Another quality parameter for foamed bitumen is defined as the half life time ($t_{1/2}$) which is the time (in seconds) during which the maximum volume of the bitumen foam is reduced by 50 %. The longer the half life time, the better quality of bitumen foam; half life time of over 15 seconds is considered superior quality (as a standard, $t_{1/2}$ should be between 8 and 15 seconds). The half life time expresses bitumen foam stability and is indirectly pro-

portionate to the expansion ratio. Both parameters mentioned depend, to a certain degree, on the type and origin of the bituminous binder, hot bitumen temperature as well as the quantity of pressurised air added and the pressure under which water is injected in the hot bitumen. The intensity and effectiveness of the foaming effect might be influenced by regulation of basic physical conditions like temperature, humidity and pressure [1, 4]. From the point of view of sufficiently homogeneous mixes, the size and arrangement of the mixing device (e.g. the injection zone size, mixing apparatus type, optimum aggregate composition, or optimal proportion of fine particles) is also important, on top of the expansion ratio.
In this regard, this paper aims to evaluate the effect of bituminous binders obtained from various locations and by various producers with different parameters (effects of the origin of crude oil) and of foaming additives on the properties of foamed bitumen and cold recycling mixes. The production, test methods and cold recycling mix assessments were carried out according to the Czech technical specifications TP 208 [5]; indirect tensile strength was monitored for 7 and 14 days' curing under predefined conditions in test specimens of 150 mm diameter compressed by static pressure. Other parameters examined were water susceptibility and stiffness modulus determined by the IT-CY method under EN 12697-26.

2 Materials and test methods applied

Within the framework of assessing the effect of bituminous binders and foaming additives on cold recycling mixes, bituminous binders from different locations and by different producers were selected. A set of test methods according to [5] was defined for the test mix assessment as such. From the perspective of the composition of the recycled mix aggregate skeleton, sorted reclaimed asphalt material of fraction 0/22 from the Středokluky mixing plant was used; extraction with determination of the content of soluble binder and a grading analysis indicated in Fig. 1. were performed. The binder content was detected as 5.6 %-wt. The optimal moisture content of the cold recycling mix for the composition as specified below was determined according to [7].

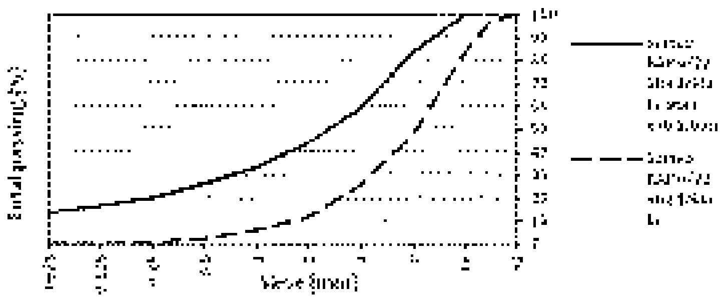

Figure 1 Granularity of reclaimed material 0/22 Středokluky

Bituminous binders of 70/100 and 160/220 penetration grade which are traditionally used in the Czech Republic were utilised in mix design and production. Three representatives of distilled bitumen 70/100 and two representatives of distilled bitumen 160/220 were assessed. In the case of the former group, the bituminous binders were based on Russian crude oil (Russian Blend); in the case of some other bitumen variants, there was a blend of several sorts of oil with a significant representation of North Sea raw material (Mittelplatte crude oil). The EM mark with some binders means “fit for emulsifying”; the crucial factor is the salt content and neutralising number of these binders. This is usually different for each sort of crude oil. Bitumen 160/220 was compared to the binder allegedly based on Venezuelan crude oil which, from the perspective of bitumen exploitation rate and quality of the resulting distilled bitumen, is among the best. Only the characteristics of softening point and penetration under 25°C were examined for all of the assessed bituminous binders always two samples for penetration and four rings for softening point. Additive A (Iterchimica; Iterfoam) is used as a foaming thixotropic agent. The additive is of a dark, oily colour and from the chemical perspective it is a combination of different amine compounds. In the case of additive B (MeadWestVaco; Evotherm), the additive is an oily liquid of amber colour, based on derivatives of fatty acid amines. The additive is intended for applications in the field of low temperature asphalt mixes. Its application as a potential foaming agent constituted an assumption, not verified as yet, with respect to improving the surface activity (which is improved by this additive). The producer declares production and laying temperature reduction for the asphalt mix by 30-45°C. The additive has volume weight of 970 kg/m^3 and flash point CoC over 204°C, [8, 9]. The value of penetration and softening point was determined for each binder, see Table 1.

Table 1 Selected properties of the bitumen binders applied

Binder	Characteristic	
	Penetration under 25°C [0,1 mm]	**Softening point [°C]**
Azalt 70/100 (Total)	83	46,8
Aqualt 160/220 EM (Total)	156	41,6
Aqualt 70/100 EM (Total)	79	47,4
70/100 EM	86	46,2
160/220 EM (Nynas)	165	40,8

The next step was optimising the foamed bitumen primarily from the perspective of the parameters it must meet. For the sake of good coating of the reclaimed material in the mix, the quantity of water applied to the foam should be optimised to keep the expansion ratio ≥ 8 and the half life time ≥ 6 s in compliance with the recommendations according to [4] for aggregate of temperature ≤ 25ºC. The expansion ratios and half life times of experimentally assessed foamed bitumen variants depending on used and examined bituminous binders are indicated in Fig. 2. What is rather interesting is the increased half life time of the foamed bitumen with foaming additive A while the half life time is ten-fold higher, thus allowing a longer mixing time. Contrastingly, and contrary to the expectations, additive B deteriorates slightly the quality of the foam, nevertheless not being primarily indicated as a foaming agent. The poor quality of the foam with the additive resulted in abandoning its application in cold recycling mixes in the future. The values of optimal quantities of water added to the bitumen according to the aforementioned parameters are indicated in Table 2.

Table 2 Quantity of water added to the bitumen during foam production

Bituminous Binder	Optimum added water [% of bitumen]
Azalt 70/100 (Total)	3,8
Aqualt 160/220 EM (Total)	1,8
Aqualt 70/100 EM (Total)	2,4
70/100 (Vialit)	1,1
160/220 (Nyanäs)	3,0
Azalt 70/100 (Total) + 0,4% additive A	3,8
Azalt 70/100 (Total) + 0,6% additive A	3,8
Azalt 70/100 (Total) + 0,4% additive B	not defined
Azalt 70/100 (Total) + 0,6% additive B	not defined

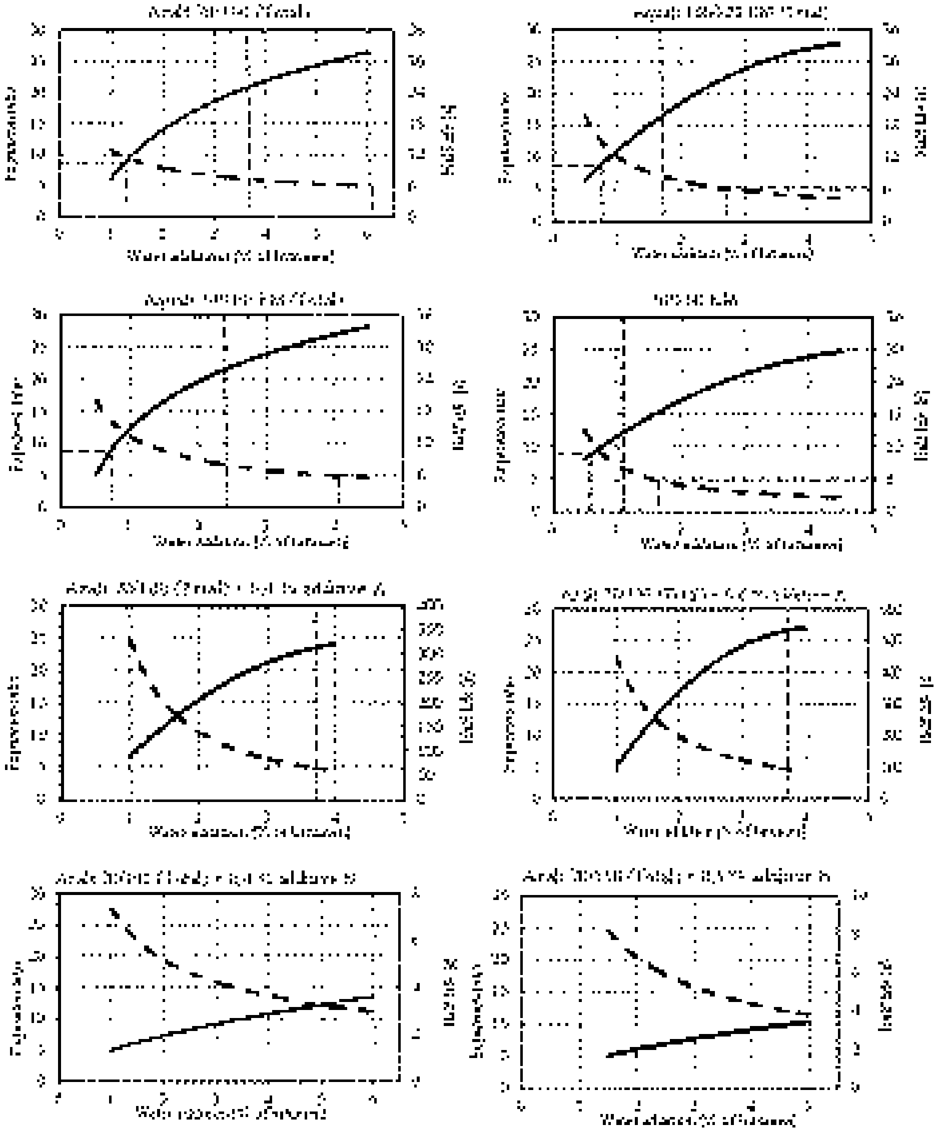

Figure 2 Foamed bitumen parameters (Expansion ratio and half life time)

Foamed bitumen was injected in the mix under the temperature of 170°C by means of the Wirtgen WLB10S laboratory equipment. The mix as such was mixed using a twin-shaft compulsory mixing unit Wirtgen WLM 30. The recycled mix obtained was subsequently put in cylindrical moulds and compacted by static pressure of 5.0 MPa. Technical conditions for cold recycling mixes [5] stipulate test specimen preparation by cylindrical moulds with 150±1 mm diameter and 200-300 mm height. The basic volumetric parameters were determined for the test specimens prepared, as well as the indirect tensile strength under [5] and stiffness modulus by means of a non-destructive test by repetitive indirect tensile stress under 15°C according to [6] and water susceptibility. Specimens were tested after 7, 14 days' dry curing under 20 ± 2°C and after 14 days of exposure to air and water.

3 Discussion of the results

The summary results of the parameters examined in the recycled mixes are indicated in Fig. 3. For indirect tensile strength and each period of curing at least four specimens were tested. Cold recycling mixes with bituminous binders of higher penetration gradation demonstrate slightly lower indirect tensile strength values from the point of view of final tensile strengths and, at the same time, dry curing of the test specimens. However, the trend is not too obvious and might be affected, with respect to the void contents achieved, by a lack of homogeneity in the reclaimed material. In contrast, the mixes containing higher penetration value bituminous binders are less susceptible to water. This indication might support the theory of superior binder coating of the aggregate. The stiffness modulus values copy the trend mentioned within the framework of indirect tensile strength. It should be stated that all mixes meet the requirements for foamed bitumen-bound recycled mixes given by [5]. The foaming agent examined did not manifest itself, i.e. it failed to affect the parameters examined and, therefore, we can only consider its application from the point of view of the possible extended mixing time which could positively influence the quality of mix coating.

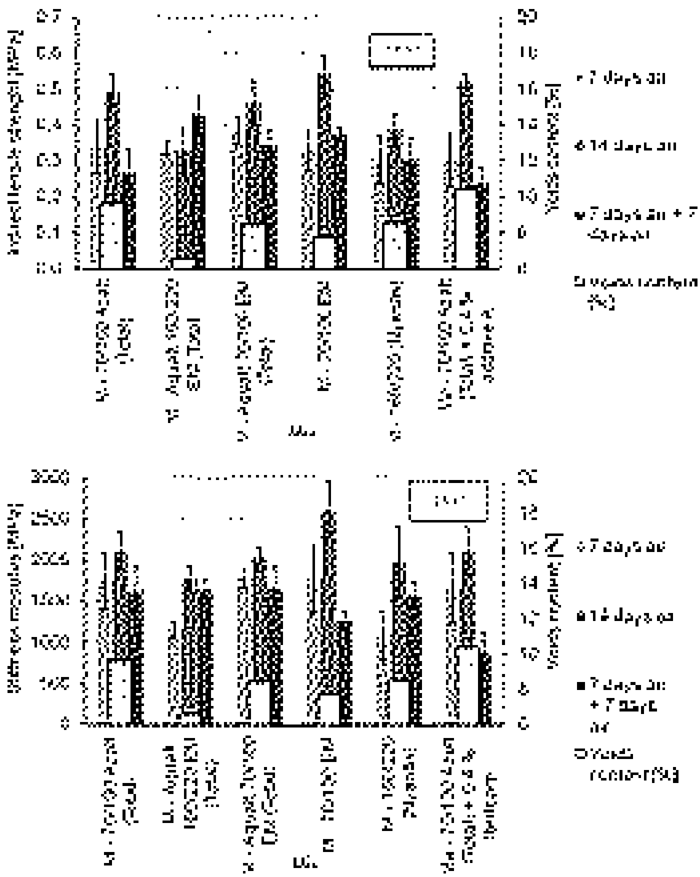

Figure 3 Parameters examined (ITS and stiffness modulus) of cold recycling mixes

4 Conclusion

The objectives of researching the relationship between bitumen properties and the characteristics of the resulting foam depending on the strength and deformation parameters of recycles mixes achieved with respect to the aforementioned test results are well understandable within the framework of the attempt to unify the design and production regulations within the European COREPASOL project. The difference in inputs in individual countries can be rather big according to the assessments obtained. The study emphasises the assessment of the effect of the bitumen origin on the cold recycling mix properties while the existing current results concerning the issue can be summarised as inconclusive at least.
From the results gained by the research it is obvious that bituminous binders of different penetration and origin usually have similar problem of satisfactory expansion ratio but very low half-life time. This finding corresponds very well with the practical site experience and affects the quality of material coating during cold recycling application. At the same time it could be shown that addition already of 0.4 % of a suitable foaming agent results in a distinctive increase in half-life showing very stable foamed bitumen. If these results are compared to cold recycled mix strength results, however, the water sensitivity of the mix with foam containing foaming agent was increased and even the strength after 7 days curing reached rather a lower value. The theoretical but hardly accountable reason could be a longer time period necessary for settlement of the bitumen coating.

Acknowledgments

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Reference

[1] Iwański, M., Chomicz-Kowalska, A.: Laboratory Study on Mechanical Parameters of Foamed Bitumen Mixtures in the Cold Recycling Technology, Procedia Engineering, Volume 57, 2013, Pages 433-442, ISSN 1877-7058.

[2] Chandra, R., Veeraragavan, A., Murali Krishnan, J.: Evaluation of Mix Design Methods for Reclaimed Asphalt Pavement Mixes with Foamed Bitumen, Procedia – Social and Behavioral Sciences, Volume 104, 2 December 2013, Pages 2-11, ISSN 1877-0428.

[3] Saleh, M.F.: Effect of rheology on the bitumen foamability and mechanical properties of foam bitumen stabilised mixes, International Journal of Pavement Engineering, Vol. 8, Iss. 2, 2007.

[4] Wirtgen: Wirtgen Cold Recycling Technology 2012 Manual, Wirtgen GmbH, first edition, Windhagen, Germany, 2012.

[5] Czech Minsitry of Transportation: Technical specification TP 208, Ministry of Transportation, Praha, 2009.

[6] ČSN EN 12697-26 Asfaltové směsi – Zkušební metody pro asfaltové směsi za horka – Část 26: Tuhost, 2012.

[7] ČSN EN 13286-2, Nestmelené směsi a směsi stmelené hydraulickými pojivy – Část 2: Zkušební metody pro stanovení laboratorní srovnávací objemové hmotnosti a vlhkosti – Proctorova zkouška, 2011, ÚNMZ.

[8] McAsphalt. http://www.mcasphalt.com/en/files/pdf/products/2Evotherm_20 Contractor.pdf (accessed May 11, 2013).

[9] Evotherm™ 3G M1 Safety Data Sheet. Notrh Charleston – South Carolina: MeadWestvaco Corporation, 2011.

INFLUENCE OF CHEMICAL CATALYSTS AND SELECTED ADDITIVES ON BEHAVIOR OF CRUMB RUBBER MODIFIED BITUMEN

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Abstract

Since several years research in the field of crumb rubber modified bitumen (CRMB) is again an important area of interest. This fact is driven by restrictions for land-filling of old tires and their energetic use as a solid fuel e.g. in cement production. At the same time some experts expect that the use of higher content of crumb rubber in bitumen can further decrease the noise reducing potential if used in suitable acoustic asphalt mixes. Last but not least using crumb rubber as a modifier creates on the market binder which would fill the application gap between distilled bitumen and PMBs. Within the research done at Czech Technical University in Prague different types of micro-pulverized rubber has been used together with chemical catalysts suitable for temporary devulcanization of the rubber. Further for improving the homogeneity and storage stability of the CRMB additional additives have been tested and their effects compared. For the bitumen samples traditional as well as performance-based tests have been done. Within the traditional test softening point, penetration and elastic recovery test should be stressed out. For performance-based testing dynamic shear rheometer has been used for running frequency sweeps for temperatures between 20 and 60°C. At the same time multiple stress creep recovery test has been done for test temperature of 60°C. Dynamic viscosity has been assessed as well. Paper summarizes gained findings and recommendations made for most suitable combinations of bitumen – CR content – catalyst and additive.

Keywords: crumb rubber modified bitumen, pulverized rubber, chemical catalysts, penetration, complex shear modulus, storage stability, elastic recovery

1 Introduction

As stated in many reports and studies it is assumed that the yearly waste production of old tires worldwide reaches more than 110 mil. tires of different type and composition. This might represent more than 7,000 kT of used rubber most of it coming from EU countries and North America. In many regions and countries different regulations or legal standards have been set how to deal with the waste of old tires. In the developed countries it is since several years forbidden put the tires on landfills (in some parts of USA slashed tires can be land filled) and other solutions are preferred. If following the European waste management strategy the most preferred solution is recycling and reuse. One area, which is for several decades understood as a potential field of crumb rubber utilization, are asphalt pavements and especially modification of bituminous binders.

Generally two technical directions are know – dry process during which crumb rubber is added directly to the asphalt mixture as a modifier and substitute to part of the finer aggregates and wet process where the bitumen is modified. The focus of this paper follows the second process which might fill the gap between distilled bitumen and polymer modified bitumen

(in performance and price). Nevertheless one of the crucial issues related to this type of bitumen modification is the homogeneity of the final crumb rubber modified binder (CRmB). Of course it is possible to produce this type of bitumen directly on an asphalt mixing plant and there are various solutions of bitumen blenders which are applied for such production. The question always prevails if this is the most suitable solution. If CRmB is produced industrially in refinery or a bitumen manufacturing plant quality control should be better and producer has to declare the properties of final product. In case of EU producer is responsible also for all necessary steps related to REACH directive. The target then is to get a binder which can be transported for longer distances and ideally can be stored on a mixing plant for a few days. I.e. homogeneous storable bitumen is required, which is usually not easy to reach because of very strong sulphur bridges in the rubber. Several approaches can be found worldwide based e.g. on polyphosporic acid or macrocyclic polymers. Usually the additive itself is not the solutions and the composite material crumb rubber-bitumen-additive works only for limited rubber content. At the same time successful techniques applied in this area practically usually are not able to be used as binders for bituminous emulsion production because of undiluted rubber particles in the bitumen composite which would most probably clog the nozzles on a spraying bar.
Based on this knowledge this paper focuses on two issues. Using special type of disintegration technique for producing grinded rubber with particles ‹1.0 mm in different grading including the assessment of effect of such rubber on bitumen performance and analyzing selected types of catalysts and additives which might help to produce a storage stable product. The process of high speed grinding (disintegration) is described e.g. in [1].

2 Assessed crumb rubber modified binder variants

The assessment of the group of experimental bituminous binders modified by crumb rubber (CRmB) involved an application of finely pulverised rubber of three grading (granularity) levels. At the same time, the effects of several catalysts were checked; these meant special organic solutions on an anhydrous basis the chemical composition and pH of which vary and are know-how protected. Catalyst K2 is neutral, i.e. with pH=7, catalyst K3 has a slightly acidic nature with pH=5. Both catalysts are based on a combination of methane, $(CH_2)_n$ and SO_2 compounds bond in a complex hydrocarbon chain. Simultaneously, an original Czech additive, Polyol was applied; according to the information available, this is a by-product of an innovative chemical recycling method for polyurethane. Last but not least, additive commercially known as Vestenamer was used which is sufficiently well established from the ROAD+ technology. This additive is a mix of linear and macro-cyclical polymers, chemically termed trans-polyoctenamer (TOR). This is applied together with crumb rubber by a content not exceeding 5 % and then mixed in the bitumen. Variations of experimentally tested crumb rubber modified binders further discussed in this paper are summarized in Table 1.
The choice for the basic bituminous binder was the standard distilled bitumen, 50/70, for which standard ČSN EN 12591 defines the interval for the softening point as 46-54°C and the penetration interval of 50-70 dmm. Such basic criteria allow an approximate assessment of the effect of the combination of the crumb rubber and individual chemical additives applied on the individual characteristics. Simultaneously, the requirements stipulated by German regulations for standardised binders of the GmB (CRmB) type can be used where GmB 25/55-50 or 25/55-55 appears to be the most appropriate for further assessment. The threshold parameters are summarized in Table 2.

Table 1 Assessed experimentally designed CRmB variants

Bitumen variant	Additives	Pulverized rubber	Bitumen composition
CR-L7_2	–	15 %; 0,8 – 1,0 mm	50/70 + 15% CR
CR-L7_2_K2 @150	Catalyst K2	15 %; 0,8 – 1,0 mm	50/70 + 15%CR + 5%K2@150°C
CR-L7_2_K2 @170	Catalyst K2 @170°C	15 %; 0,8 – 1,0 mm	50/70 + 15%CR + 5% K2 @170°C
CR-L7_2_K3 @150	Catalyst K3	15 %; 0,8 – 1,0 mm	50/70 + 15%CR + 5%K3@150°C
CR-L7_2_K3_P	Catalyst K3 + Polyol	15 %; 0,8 – 1,0 mm	50/70 + 15%CR + 5%K3 + 1%Polyol
CR-L7_2_K4 @150	Catalyst K4	15 %; 0,8 – 1,0 mm	50/70 + 15%CR + 5%K4@150°C
CR-L7_2_K4 @170	Catalyst K4	15 %; 0,8 – 1,0 mm	50/70 + 15%CR + 5%K4@170°C
CR-L7_2_K4_P	Catalyst K4 + Polyol	15 %; 0,8 – 1,0 mm	50/70 + 15%CR + 5%K4 + 1%Polyol
CR-L7_2_V	Vestenamer	15 %; 0,8 – 1,0 mm	50/70 + 15%CR + Vestenamer (4:100)
CR-L8_2_K4 @150	Catalyst K4	15 %; 0,5 – 0,8 mm	50/70 + 15%CR + 5%K4@150°C
CR-L9_2_K4 @150	Catalyst K4	15 %; 0,1 – 0,3 mm	50/70 + 15%CR + 5%K4@150°C

Table 2 Target values of German specifications for terminal blended CRmBs

Characteristic	Unit	GmB 25/55-50	GmB 25/55-55
Penetration	dmm	25-55	25-55
Softening point	°C	›50	›55
Elastic recovery	%	›50	›50
Complex shear modulus G* @ 60°C, 1,59 Hz, 2 mm gap	Pa	≥ 6000	≥ 8000
Phase angle δ @ 60°C, 1,59 Hz, 2 mm gap	°	‹65	‹65

3 Selected test methods

Standard empirical testing and test methods were selected to assess selected functional characteristics within the evaluation of the impact of the micro-pulverised rubber. Empirical characteristics:
- softening point determination by means of the ring and ball method (EN 1427);
- determination of needle penetration under 25°C (EN 1426);
- determination of elastic recovery under 25°C (EN 13397);
- storage stability test; 72±1 h and temperature of 180°C (EN 13399).

Functional characteristics:
- determination of the complex shear modulus G* and phase angle δ under 60°C and under 40°C;
- frequency sweep for G* and δ with subsequent plotting of the master curve for the reference temperature of 20°C;
- dynamic viscosity determination (EN 13302).

The elastic recovery determination followed the EN 13397 technical standard. The principle of the test is extending the sample in a water bath under 25°C to the bitumen fibre length of 20.0 cm and subsequent observation of the sample shrinking for 30 minutes while the fibre is cut off at the beginning. The test provides the recovery value expressed as a percentage. The measurement of dynamic viscosity is based on the degree of the sample's resistance to the stress caused under the selected angular velocity. A defined torsion stress is applied to the sample to obtain the relative resistance to spindle revolution. The measurement is taken under various test temperature. The condition is important primarily with modified bitumi-

nous binders or in cases where bituminous binders are improved or modified by various additives. In accordance with the findings and recommendations of the U.S. SHRP program, measurements for distilled binders should be taken for the temperature of 135°C which is considered a suitable representative for the determination of workability level of the sample in question. The standard stipulates a rotational spindle viscometer as the measuring apparatus and ranges for the shear rate ($1 – 10^4$ s^{-1}) and dynamic viscosity ($10^{-2} – 10^3$ Pa·s) under temperatures ranging from 40°C to 200°C. The temperature regulation equipment must be capable of regulation with the precision of ± 0.5°C.
The determination of complex shear modulus G* and phase angle δ of bituminous binders using the dynamic shear rheometer (DSR) is governed by technical standard EN 14770. Simultaneously with the measurement of dynamic shear, the viscose and elastic behaviour of the binder can also be examined through the determination of the complex shear modulus and phase angles under varying temperatures and frequencies which, together, cover a broad spectrum of possible conditions to which the bituminous binder might be exposed.
The determination of G* and δ in the oscillation test is usually carried out for a temperature range of 20-100°C. A specific stress frequency or a pre-defined frequency spectrum is selected. To obtain relevant results, the linear area of visco-elastic behaviour must be defined, i.e. in the regime where the test is conducted with controlled stress; the constant shear stress for the test must be specified. In this case, we used the previous findings of the CTU Faculty of Civil Engineering, for bituminous binders 50/70 and 70/100. The shear stress of τ=2000 Pa is considered safe and appropriate.
Additionally using the time-temperature superposition principle – TTS, the values obtained from measurements under various temperatures and load frequencies may be summarised (transposed) in a single characteristic known as master curve for the selected reference temperatures which, in the case of the results presented below, amounted to 20°C.
Measurements taken under invariable shear stress within the interval of the selected stress frequency called the "frequency sweep"(FS) form a common test. The test is intended for verification of the bituminous binder behaviour under varying temperatures and frequencies, or stress durations. The test frequencies were chosen from a range of 0.1 – 10 Hz for the binders examined. According to the previous findings and conclusions of the U.S. SHRP, the value of 1.59 Hz is very important since, according to the Van der Poel's nomograph, it corresponds with the shear effect of traffic load at 90 km/h, or load duration of 0.13 s.

4 Experimental results and discussion

The results are further divided into several logical groups which assess either the effect of the catalyst or the additive applied, or the influence of the pulverised rubber or the temperature chosen for the production of the CRmB binder.

4.1 Influence of chemical catalyst applied in pulverized rubber

Samples of experimentally prepared CRmB binders with the application of different catalysts were compared, namely versions K2, K3 and K4 to the quantity of 5 %-wt. together with pulverised rubber to the quantity of 15 %-wt. in the bitumen. The individual binder variants were prepared under processing temperature of 150°C. Bituminous binder 50/70 modified by 15 %-wt. of crumb rubber is used as the reference sample.
Out of the three catalysts tested, catalyst K4 affects the tested parameters best. Catalysts K2 and K3 cause slightly less significant decrease of the bituminous binder penetration where they fail to meet the requirements for 50/70; however, they meet the requirements for GmB binders. Analogously, from the perspective of the softening point, catalysts K2 and K3 result in higher values while the use of catalyst K4 in combination with 15 % rubber basically remains unchanged. All three binders meet the requirement for GmB 25/55-50. The option with catalyst

K2 achieved the best results in the storage stability test where the difference between the lower and upper softening point is 4.5°C. The elastic recovery results demonstrate a poorer effect of catalyst K4 where the value only amounts to about 12 %. In general, none of the binders assessed meets the requirement stipulated in the German regulations for GmB binders.
The dynamic viscosity test with a focus on 20 rpm (the reference velocity considered by the American standards) shows a positive impact of catalyst K4 under both 150°C and 135°C. It is obvious that particularly catalyst K3 significantly increases the dynamic viscosity value primarily under 135°C. The overall course of the viscosity (flow curve) in the thermal range of 100°C to 150°C demonstrates the best results for the option with catalyst K4 within the framework of CRmB binder comparison.

Table 3 Empirical characteristics of assessed CRmB binders

Bitumen variant	Penetration [0.1 mm]	Softening point [°C]	PI [-]	Storage stability [°C]	Elastic recovery [%]
CR-L7_2	79	49.7	-1.0	9.9	8.8
CR-L7_2_K2 @150	49	57.4	-0.1	4.5	30.3
CR-L7_2_K3 @150	47	58.8	0.4	7.9	34.2
CR-L7_2_K4 @150	54	51.4	0.1	8.2	11.9

Table 4 Dyamic viscosity values of assessed CRmB binders

Bitumen variant	Dynamic viscosity @ 6.8 s^{-1} (20 rpm); [Pa·s]	
	135°C	150°C
CR-L7_2	4.09	1.54
CR-L7_2_K2 @150	5.54	1.28
CR-L7_2_K3 @150	7.10	1.74
CR-L7_2_K4 @150	3.05	1.00

The complex shear modulus values under the two temperatures chosen and frequency of 1.59 Hz are greatly affected primarily when catalyst K3 was applied, where CRmB binder reach noticeably higher values of G* than the bituminous binders versions with the remaining two catalysts or CRmB with no catalyst applied. In this context, we can note that the three versions achieve almost identical results.

Table 5 Complex shear modulus values for selected CRmB binders

Bitumen variant	60°C	40°C
	Complex shear modulus G* [Pa]	
CR-L7_2	3 309	66 742
CR-L7_2_K2@150	n.a.	73 377
CR-L7_2_K3@150	22 926	131 374
CR-L7_2_K4@150	1 480	48 300

From the perspective of the flow curves (see Figure 1), particularly a significant difference between the influence of catalysts K3 and K4 is obvious. At 130°C, the viscosity difference is double-fold, under 100°C the flow curve values differ even more. From the point of view of the above stated, the influence of catalyst K4 is either similar to low-viscosity additives or helps improve dissolution of the rubber particles in the CRmB binder composite. From the perspective of the course of dynamic viscosity, catalyst K2 appears neutral when compared to the no-catalyst CRmB version – no effect whatsoever.

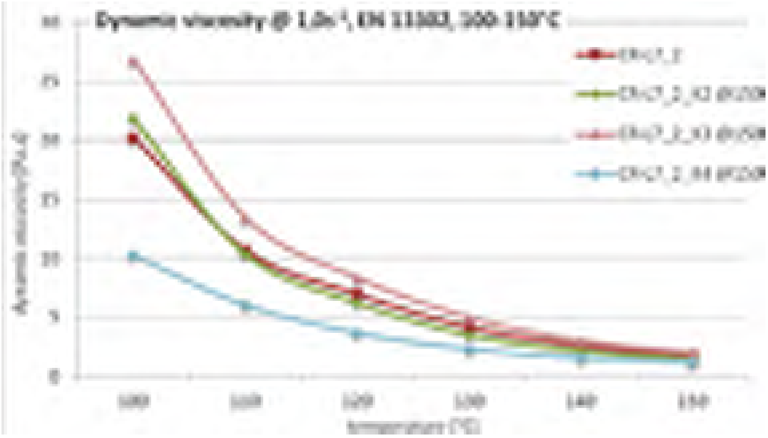

Figure 1 Flow (viscosity) curves for CRmBs with different catalysts

The aforementioned assessment of the complex shear modulus for selected temperatures and frequencies is also obvious in the case of the master curves prepared. The CRmB option with catalyst K3 records higher stiffness within the entire frequency interval – even if the conversion using the TTS principle is applied. The values of all binders assessed subsequently even out in the highest frequency interval, i.e. under 20°C and the 1-10 Hz spectrum. Based on this, we could infer that the CRmB binder with catalyst K3 will have the lowest thermal susceptibility value. In contrast to that, the course of the CRmB binder with catalyst K2 values in the smallest frequency interval (from 10^{-4} Hz) is interesting; the G* values decrease significantly. This is supported by the phase angle course where the values fluctuate slightly. It should also be pointed out in relation to the course of the phase angle master curve that the only version of CRmB binder with catalyst K3 follows a non-standard course and reaches low values which indicate prevailing elastic behaviour in the interval of 10^{-5} – 10^{-1} Hz. The reason for such course is currently difficult to explain despite repeated measurement sessions. Such binder would nevertheless have excellent fatigue resistance.

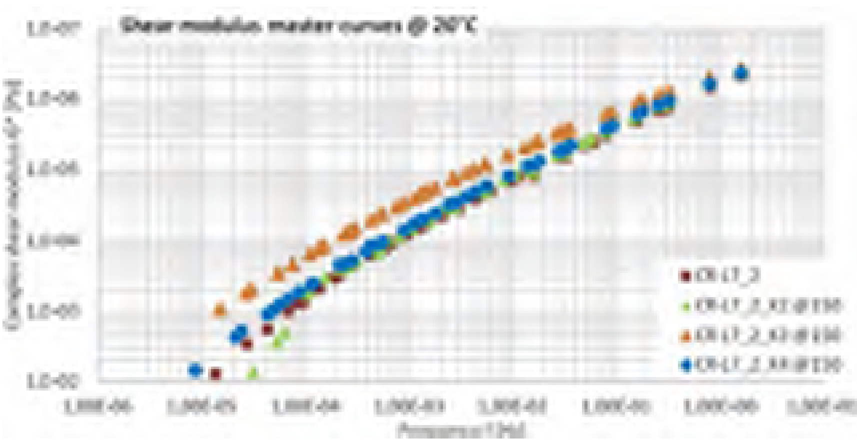

Figure 2 Complex shear modulus master curve for CRmBs with different catalysts

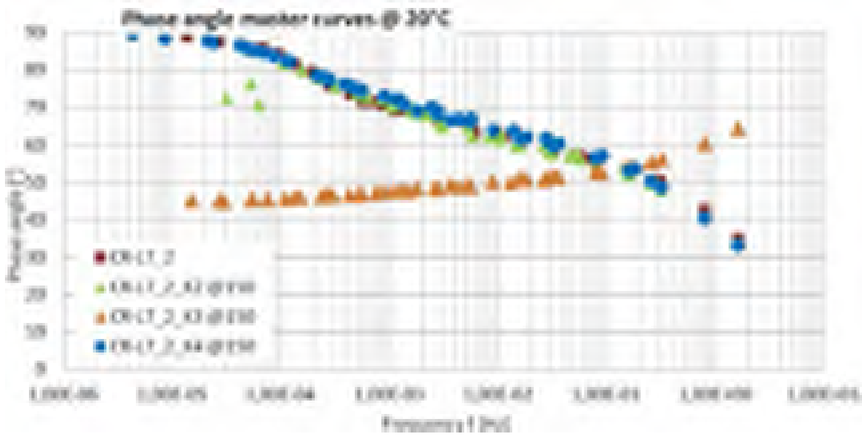

Figure 3 Phase angle master curve for CRmBs with different catalysts

4.2 Influence of pulverized rubber grading (granularity)

In this group of assessed binders, three levels of pulverised rubber grading with the same catalyst (K4) are compared; the catalyst is always applied to the same quantity of 5 %-wt. of pulverised rubber. The same distilled bituminous binder is used for all three CRmB variants; the quantities of pulverised rubber in the binder are identical, i.e. 15 % by mass. Again, CRmB with bitumen 50/70 with 15 %-wt. pulverised rubber of 0.8-1.0 mm grading is chosen as the reference binder.

Out of the three fractions of crumbed (pulverised) rubber applied, the binder with rubber of the 0.8-1.0 mm a 0.1-0.3 mm grading achieves the least effect on empirical properties. The 0.5-0.8 mm grading caused a significant decrease of CRmB binder penetration; in this case the option fails to meet the requirements for GmB 25/55-50(55). In the remaining two cases, the requirements from the perspective of both penetration and the softening point are met. Contrastingly, the noticeable increase of the elastic recovery value is interesting in the case of CRmB with 0.5-0.8 mm pulverised rubber. When the three gradings are compared no clear trend can be detected. From the perspective of the storage stability test, the option with the least grading (finest rubber) where the difference between the upper and lower softening points is 4.6°C (acceptable from the point of view of the requirements for polymer-modified bitumen) records the best results. The elastic recovery results point to an inferior effect of the 0.8-1.0 mm fraction.

The dynamic viscosity test focusing on 20 rpm shows that binders with rubber of 0.1-0.3 mm and 0.8-1.0 mm grading scored better. Even in this case, slightly illogically, the medium grading applied stands out. The overall course of viscosity in the thermal range of 100°C to 150°C demonstrates poorer results for the option with 0.5-0.8 mm fraction applied.

In the case of comparing the pulverised rubber grading effect on the complex shear modulus, the difference is obvious and the crumb rubber grading 0.5-0.8 mm achieves significantly better results. It should also be noted that for the options with rubber grading of 0.8-1.0 mm the application of catalyst K2 results in complex modulus value decreasing by 30-50 %. Contrastingly, the CRmB version with catalyst K4 and rubber grading of 0.1-0.3 mm achieves values that are basically comparable to the reference CR-L7_2 option.

Table 6 Empirical characteristics of assessed CRmB binders

Bitumen variant	Penetration [0.1 mm]	Softening point [°C]	PI [-]	Storage stability [°C]	Elastic recovery [%]
CR-L7_2	79	49.7	-0.1	9.9	8.8
CR-L7_2_K4 @150	54	51.4	-0.5	8.2	11.9
CR-L8_2_K4 @150	20	74.5	1.5	10.5	61.8
CR-L9_2_K4 @150	50	55.7	0.1	4.6	23.6

Table 7 Dyamic viscosity values of assessed CRmB binders

Bitumen variant	Dynamic viscosity @ 6.8 s^{-1} (20 rpm); [Pa·s]	
	135°C	**150°C**
CR-L7_2	4.09	1.54
CR-L7_2_K4 @150	3.05	1.00
CR-L8_2_K4 @150	13.75	4.50
CR-L9_2_K4 @150	2.06	1.03

Table 8 Complex shear modulus values for selected CRmB binders

Bitumen variant	60°C	40°C
	Complex shear modulus G* [Pa]	
CR-L7_2	3 309	66 742
CR-L7_2_K4@150	1 480	48 300
CR-L8_2_K4@150	81 200	555 000
CR-L9_2_K4@150	3 400	75 000

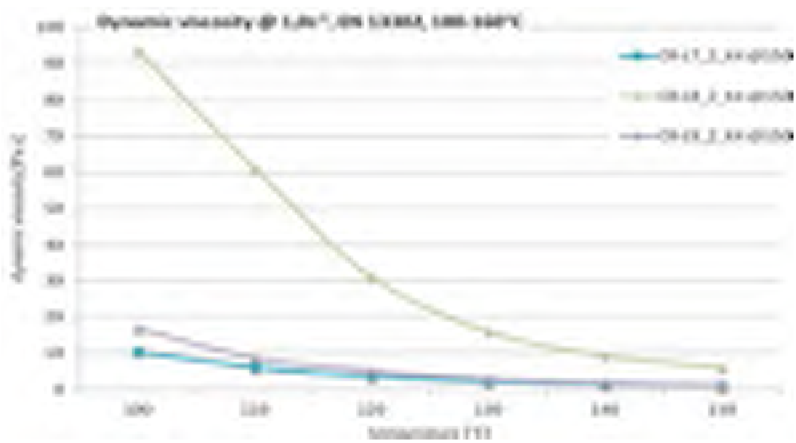

Figure 4 Flow (viscosity) curves for CRmBs with pulverized rubber of different grading

The assessment of the dynamic viscosity flow curve courses for the selected shear rates shows a significant difference between the binder with rubber of 0.5-0.8 mm grading and the remaining two versions. At 100°C the viscosity increases basically six-fold; nevertheless, even at 140°C the difference in viscosity is almost quadruple. Analogously to the complex shear modulus, the reason for the change in CRmB binder behaviour if this specific pulverised rubber fraction is applied and if all other conditions of bituminous binder sample preparation and heating remain the same is interesting and difficult to explain.

Similarly to the individual G* values determined for 40°C and 60°C, the master curve of CRmB with rubber of 0.5-0.8 mm grading reaches noticeably higher complex shear modulus values than the remaining fractions. In the field of very low frequencies (this corresponds with values of the complex shear modulus under higher temperatures), the difference even amounts to two orders. Only from the frequency of about 1.0 Hz the complex shear modulus values come closer together. At the same time, the curve for bituminous binders with rubber grading of 0.5 to 0.8 mm is flatter which means lower thermal susceptibility of the bituminous binder which can be expressed through the slope of the master curve. When the two remaining bituminous binders are compared it is obvious that from the perspective of the complex shear modulus and the expected resistance to deformation effects, the CRmB binder version with the finest pulverised rubber grading records the lowest values. Based on this comparison of the characteristic, very fine pulverisation unfortunately appears to be less appropriate. This is subsequently supported by the phase angle values where the master curve of the same binder reaches the highest δ values and, therefore, the viscose component of the complex shear modulus prevails over a great proportion of the frequency interval; in comparison to binder CR-L8_2 we can conclude on a lower fatigue life expectation. The aforementioned bituminous binder with pulverised rubber of 0.5-0.8 mm, in contrary, demonstrates a rather elastic character.

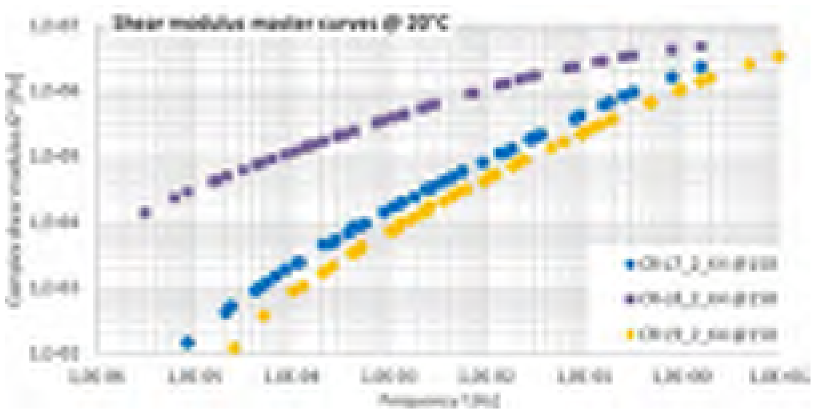

Figure 5 Complex shear modulus master curve for CRmBs with pulverized rubber of different grading

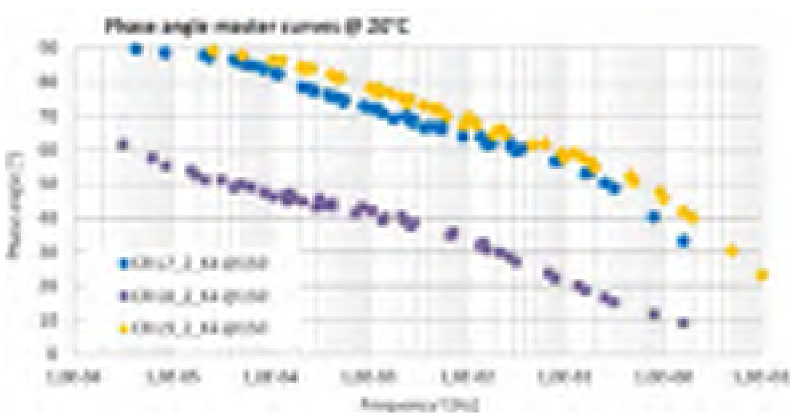

Figure 6 Phase angle master curve for CRmBs with pulverized rubber of different grading

The assessment of the course of master curves for the CRmB binders examined with the determination of the influence of the pulverised rubber grading, in accordance with the remaining results, it is obvious that the best values of the complex shear modulus by far are achieved by CRmB with rubber of 0.5-0.8 mm grading; for small frequencies, the differences are adequate to 1-2 decimal orders. At the same time, it is evident that this version of CRmB binder scores lowest on thermal susceptibility. Within the framework of the assessment of the remaining two versions, the binders are similar with minimum impact of the grading of the pulverised rubber applied.
In the case of phase angle analysis, the qualitative parameters of CRmB binders with rubber of 0.5-0.8 mm grading are confirmed. Within almost the whole frequency interval, the binder demonstrates prevailingly elastic behaviour which predetermines its very good resistance to fatigue and, simultaneously and in combination with higher G* values, high resistance to permanent deformations.

5 Conclusion

The comparisons conducted and divided on the basis of the possible influences and comparisons of multiple CRmB binder versions reveal certain tendencies in some cases. From the perspective of crumb rubber granularity, it was rather surprisingly demonstrated that particularly fraction 0.8-1.0 mm is suitable for use within the framework of determining dynamic viscosity and empirical properties. Contrastingly, rheological properties are best affected by the 0.5-0.8 mm granularity, to a lesser degree even by the finer grading of 0.3-0.5 mm. However, in this case it is possible that the results are affected by the proportion of the granulate grading and the gap between oscillation plates in the test apparatus utilised. Due to that, for instance the German technical regulations concerning GmB binders stipulate a 2 mm gap even for the geometry of PP25 to eliminate any possible influence of undissolved rubber granulate particles. Not even the choice of a suitable catalyst is absolutely clear. Catalyst K4 has a positive effect on viscosity and basic properties of bituminous binders; in contrast to that, catalyst K3 modifies elastic recovery and complex shear modulus. Catalyst K2 appears to be the most appropriate alternative to reduce the difference in softening points after the storage stability test.

Acknowledgments

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Reference

[1] Valentin, J., Benes, J., Karra´a, G., El-Dajani, N.: Assessment of potential for bituminous binders modified by micro-grinded rubber. 17th IRF World Meeting and Exhibition, Riyadh, Saudi Arabia, 2013.

[2] Willis, J. R., Plemons, C., Turner, P., Rodezno, C., Mitchell, T. (2012): Effect of ground tire rubber particle size and grinding method on asphalt binder properties. National center for asphalt technology, Auburn University, Auburn, Alabama.

[3] Marcant, B., Martin, J.-V. (2009): A New Technology of Rubber Modified Bitumen Meeting Performance and Environmental Requirements. Proceedings of Conference Asfaltové vozovky 2009, Ceske Budejovice.

5 RAIL VEHICLE-TRACK INTERACTION

CETRA[2014] 28–30 April 2014, Split, Croatia
3rd International Conference on Road and Rail Infrastructure

TRACK-STRUCTURE INTERACTION ANALYSIS USING FE MODELLING TECHNIQUES

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Abstract

With the growth in both High Speed and Light Rail infrastructure projects worldwide there is a general requirement for accurate modelling of the interaction of the track with respect to any supporting bridge structures, and in particular, to ensure that any interaction between the track and the bridge as a result of temperature and train loading is within specified design limits. To accurately assess track-structure interaction effects nonlinear analyses are required to investigate thermal loading on the bridge deck, thermal loading on the rail if any rail expansion devices are fitted, and vertical and longitudinal braking and/or acceleration loads associated with the trainsets. For a complete rail track assessment, dynamic effects caused by the passage of trains that affect the structure itself must also be considered. The paper will describe how rail track analysis for both high speed and general trainsets can be carried out according to the Union Internationale des Chemins de fer (International Union of Railways) UIC774-3 Code of Practice [1] and Eurocode 1 [2] with particular reference to LUSAS [3] Rail Track-Structure Interaction and Interactive Modal Dynamics analysis software applications. Automated modelling techniques and results and graphing capabilities will be described. Projects either built or under construction and on which the software has been used to good effect are described and cited.

Keywords: track-structure interaction, bridge, train, rail, UIC774-3

1 UIC774-3 Code of Practice / Eurocode 1

According to the UIC774-3 Code of Practice and its incorporation into the Eurocodes [2], the track-structure interaction effects should be evaluated in terms of the longitudinal reactions at supports, rail stresses induced by the temperature and train loading effects in addition to the absolute and relative displacements of the rails and deck. To accurately assess the behaviour these interaction effects should be evaluated through the use of a series of nonlinear analyses where all thermal and train loads are taken into account. Loading to consider will include:
- thermal loading on the bridge deck;
- thermal loading on the rail if any rail expansion devices are fitted;
- vertical loads associated with the trainsets;
- longitudinal braking and/or acceleration loads associated with the trainsets.

The interaction between the track and the bridge is approximated in the UIC774-3 Code of Practice by a bilinear relationship. The resistance of the track to the longitudinal displacements for a particular track type is a function of both the relative displacement of the rail to the supporting structure and the loading applied to the track. Application of train loads increases the resistance of the track to the relative displacements where these train loads are present but is unchanged for all other locations.

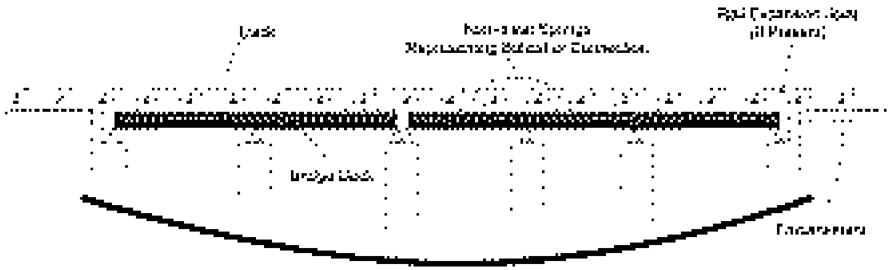

Figure 1 Representation of structural system for evaluation of interaction effects to UIC-774-3

The values of displacement and resistance to use in these bilinear curves are governed by the track structure and maintenance procedures adopted and will be specified in the design specifications for the structure. Typical values are listed in the Code of Practice for ballast, frozen ballast and track without ballast for moderate to good maintenance.
Figure 1 illustrates the structural system that needs to be considered for the interaction analysis. According to the UIC774-3 Code of Practice there is no requirement to consider a detailed model of the substructure (bearing-pier-foundation and bearing-abutment-foundation systems) when 'standard' bridges are considered, instead this can be modelled simply through constraints and/or spring supports that approximate the horizontal flexibility due to pier translational, bending and rotational movement. However, Rail Track-Structure Interaction analysis software does allow analyses to be carried out where the bearing and the pier/abutment-foundation are explicitly modelled.

2 Modelling with Rail Track Analysis Software

Rail Track-Structure Interaction analysis software provides the means to create a finite element model, analyse it, and conduct a bridge/track interaction check in accordance with the UIC774-3 Code of Practice. It can be used for single deck structures as well as for long, multi-decked, multiple span viaduct structures typical of the Gyeongbu high speed railway in South Korea, and on which it has been widely used.
Rail track and bridge interaction models are built automatically from data defined in a Microsoft Excel spreadsheet. This spreadsheet comprises a number of worksheets that relate to particular aspects of the modelling and cover: Number of Decks, Tracks and Embankment Lengths, Structure Definition, Geometric Properties, Material Properties, Interaction and Expansion Joint Properties, and Loading to be used. The number of decks that can be analysed is effectively unlimited. Either one or two tracks can be modelled and for two tracks, one will take the braking load of a trainset and the other will take the acceleration load of a separate trainset. Lengths of the embankments to either side of the structure need to be sufficiently long to allow the trainset loading to be placed within the model and, according to the UIC774-3 Code of Practice, should be at least 100 m.
For each deck the modelling spreadsheet allows the definition of the left pier/abutment, up to eight internal piers and the right pier/abutment, each with their own support / bearing characteristics. These can include the physical modelling of the piers if specified. The geometric and material properties for the rail and deck components are defined on separate worksheets. Mass density is not used in the analysis but is provided to allow the separate solution with self-weight and for it to be combined with the thermal/train loading effects covered in these types of analyses. The main bilinear interaction effects for the track/bridge interaction are defined in the Interaction and Expansion Joints worksheet along with additional properties associated with the rail/track. These include the eccentricity between the rail/slab and the presence of any rail expansion joints.

The temperature effects in the rails for a continuously welded rail (CWR) track do not cause a displacement of the track and do not need to be considered (UIC774-3 Clause 1.4.2). For all other tracks the change in temperature of the bridge deck and rails relative to the reference temperature of the deck when the rail was fixed needs to be considered in accordance to the code of practice and design specifications. To achieve this, temperature values for the deck and for the rails are defined along with as many rail/train loads as required to completely describe the loading regime.
Train loading is defined in terms of the type (braking, acceleration or vertical loading), track, position and magnitude. Complex loading patterns and parametric loading can be defined to investigate multiple positions of the trainsets with minimum effort. At model creation time a user-specified element length (in accordance with the limitations in UIC774-3) is used to define the embankment and bridge features of the model with all of the analyses generated automatically.

3 Analysis

When running an analysis, deck and rail track temperature loading can be considered in isolation for subsequent analysis of multiple rail configurations, or a full analysis can be carried out considering the combined temperature in the deck and rail track plus trainset loading. Because the response of the ballast and/or track restraining clips is nonlinear a nonlinear analysis is always needed. During an analysis the Track-Structure Interaction analysis software automatically updates the material properties associated with the track/structure interface based upon the position of the train or trains – a key requirement to get accurate results. For a ‘total’ rail track/interaction assessment, dynamic effects caused by the passage of trains that affect the structure itself could also be considered.

4 Results viewing / processing

Results are produced in Microsoft Excel spreadsheet format or a propriety software results format. Separate worksheets within the results spreadsheet contain results for specific areas of interest. These worksheets include:
- raw results data in summary, graph and tabular form for each track and deck component;
- envelopes of raw track and deck data in summary, graph and tabular form for combinations of temperature and trainset rail loading;
- tables of railbed and bridge displacements;
- tables of longitudinal reactions;
- tables of rail stress values.

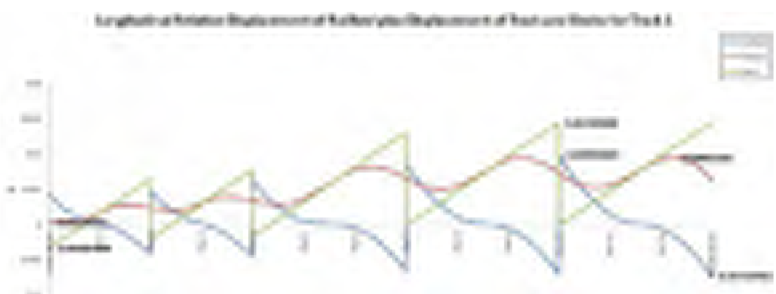

Figure 2 Envelope of axial stresses in a rail track from temperature loading

The tabular results provide summaries to allow the quick determination of which analysis is causing the worst effects for each of the checks that need to be carried out to the UIC774-3

Code of Practice which include: Relative Displacement between Rails and Deck; Longitudinal Relative Displacement between Ends of Decks (axial, end rotation and total effects); Vertical Relative Displacement between Ends of Decks; Longitudinal Reactions; and Axial Rail Stress. A sample results summary table and envelope of relative railbed plus track / deck longitudinal displacements in a rail track is shown in Figure 2.

5 Key values for checking to UIC774-3

5.1 Peak relative railbed displacement

For a continuously welded rail (CWR) track the typical criteria to be met for the relative railbed displacements is quoted in Clause 1.5.3 of UIC774-3 which states that: "The maximum permissible displacement between rail and deck or embankment under braking and/or acceleration forces is 4mm.". To permit checking of these criteria railbed displacements are included in a Track results worksheet. These are output in the form of the maximum and minimum values which are reported in the summaries at the top of the sets of results, the values over the structure graphed in the top chart and the individual values along the length of the track in tabular form. Summary tables are also created which indicate which track and location is associated with the peak value.

5.2 Peak longitudinal reactions at the abutments

These are provided in a Longitudinal Reactions Check worksheet and show both the position at which the trainset(s) is/are when the peak longitudinal reaction occurred as well as the peak reaction and where it occurs.

5.3 Peak axial rail stresses

For a continuously welded rail track with UIC 60 rails the typical criteria to be met for the rail stress are quoted in Clause 1.5.2 of UIC774-3 which states that "The maximum permissible additional compressive rail stress is 72 N/mm^2" and "The maximum permissible additional tensile rail stress is 92 N/mm^2." These criteria rail axial stress values are also included in the Track results and summary worksheets and are of the same form as presented for the relative railbed displacements.

6 'Simplified' and 'Complete' UIC 774-3 analysis methods

For a computer analysis, according to UIC 774-3 (Clause 1.7) two different methods of analysis can be used, each giving a different level of accuracy. These are termed 'simplified' analysis and 'complete' analysis. Simplified analysis considers the temperature, and the longitudinal and vertical train effects separately and permits them to be combined to get a total effect. Simplified analysis is actually quite conservative because it assumes that superposition of results is valid for sets of nonlinear analyses when it is not – it is only really valid for linear analysis. As a result combining 'simplified' analysis results can lead to an over-estimate of the rail stresses. Complete simultaneous analysis considers the temperature and longitudinal and vertical train effects simultaneously. As a result there are accuracy benefits in using Rail Track-Structure Interaction analysis software to carry out a complete simultaneous analysis, rather than using software that combines results from simplified analyses.
To illustrate the differences obtained, rail stress results due to braking and acceleration on the two tracks crossing Hwashil viaduct on the Gyeongbu line in South Korea are shown in Table 1. A simplified analysis carried out in LUSAS considered the temperature and the longitudinal and vertical train effects separately before combining them to get a total effect. Then,

a ‘complete’ simultaneous analysis was carried out for the same viaduct using the Rail Track-Structure Interaction analysis software option. The ratio of the ‘Simplified’ to the ‘Complete’ analysis shows the over-estimate involved. The key issue with the separate analysis approach is the ability for the track resistance to be overestimated by the combination of the two nonlinear analyses and potentially cause the rail stresses to be overestimated also. A study of the Hwasil viaduct by Lee et al [4] showed similar results.

Table 1 Comparison of peak compressive rail stresses for Hwashil viaduct for ‘Simplified’ and ‘Complete’ UIC774-3 analysis methods

Trainset Loading Type	‘Simplified’ : Separate Nonlinear Analysis Of Thermal And Train Loading [N/mm²]	‘Complete’: Nonlinear Thermal And Train Loading With Material Change [N/mm²]	Over-Prediction Ratio ‘Simplified’ / ‘Complete’
Track 1 (Braking)	94.99	79.08	1.2
Track 2 (Accelerating)	103.66	92.58	1.12

Comparison of the results for the separate and complete analyses shows that the peak compressive stress for the separate analysis is 1.2 times that of the complete analysis for track 1 and 1.12 times that for track 2. It should be noted however that from other studies undertaken (that are outside the scope of this paper) the separate analysis method can give an apparent increase in track resistance of up to 1.6 times that of the loaded track due to the combination of the nonlinear results. One overall conclusion is obvious from the analyses: when a combined thermal and train loading from a separate analysis gives interaction forces that exceed the stated yield resistance then the separate analysis method will potentially over predict the rail stresses unless the loaded track yield surface is reduced by the mobilised track resistance over the extent of the train loading.

7 Rail Track Analysis in practice

7.1 High speed rail

In addition to reported uses on all bridges on the Gyeongbu High Speed Railway 2nd phase (between Daegu and Busan), Saman Engineering Corporation used Rail Track Analysis software for preliminary design work on the Honam High Speed Railway on behalf of its client the Korea Rail Network Authority. This high speed railway, when complete, will link South Korea’s capital city, Seoul, with Mokpo, a southern port city in South Jeolla Province.

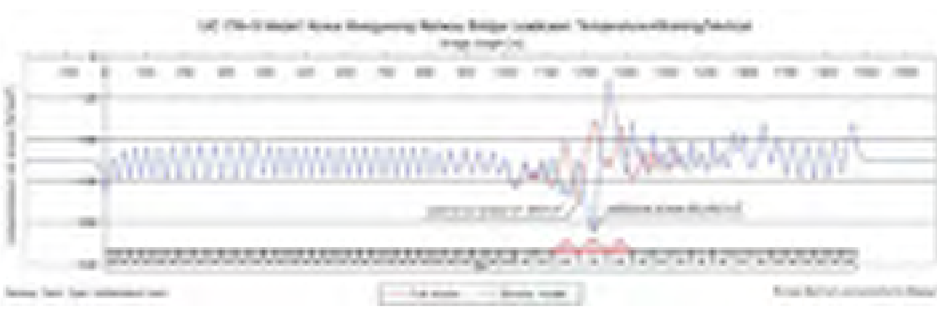

Figure 3 Compressive stress in the rails from temperature and braking loading for Mangyeong River Crossing, Honam High Speed Railway, South Korea.

The Mangyeong River crossing, one of many structures on the new route, has a length of 1,875 m and comprises a total of 50 spans of varying length and construction type. Three steel box framed spans of 60/75/60 metres over the river are flanked by steel girders of 50 m span, and then by various numbers of 35 m and 30 m pre-stressed concrete box section spans for the remainder of the crossing's length. Saman carried out a rail track/structure interaction analysis to evaluate axial stresses in the rails due to acceleration and braking forces caused by passing trains. Induced track displacements relative to the bridge deck were checked and found to be within the specified design limits.

7.2 Light rail

The Dallas Area Rapid Transit (DART) light rail system in the United States is a good example of the use of Rail Track Analysis on a light rail project. DART, as a whole, currently comprises 85 miles (137 km) between its four lines – Red, Blue, Green and Orange. US Consultant Gannett Fleming was responsible for the design of 7 new structures along a 4.75 mile (8 km) extension to the Blue route, which was completed in December 2012. The longest of these, a causeway bridge over Rowlett Creek, is a 28 span, 2,565 foot (782 m) long, pre-cast concrete beam structure. This, and the 11 span, 1054 foot (321 m) long KCS bridge consisting of both precast concrete beams and a long-span steel through girder were both analysed using Rail Track Analysis software to assess track-structure interaction effects and verify key values were within acceptable limits.

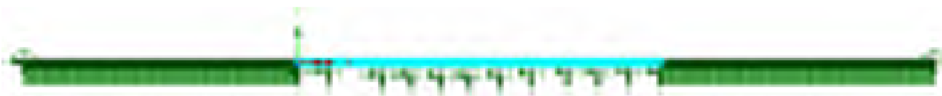

Figure 4 Rail Track Analysis Model for KCS Bridge, DART Blue Line extension, USA.

8 Dynamic analysis

For a 'total' rail track/structure analysis assessment, and depending upon the requirements of a design code, dynamic effects caused by the passage of trains across a structure may also need to be considered. This could take the form of analysing the response of the structure over time based on contributions of a number of natural frequencies (time response) or the response of the structure to dynamic loading at known frequencies (forced response). Using linear Interactive Modal Dynamic superposition techniques, rapid solutions to moving load analyses of trainsets over time at varying speeds can be achieved with output such as time histories and peak summaries, secondary response spectra, and animated visualisations being obtained. Alternatively step-by-step dynamics (full transient analysis) could be done to examine the response of the complete structure over time taking all effects (including nonlinear effects) into account. Eurocode EN 1991-2:2003 Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges [2] provides good guidance via a flowchart for determining when a static or dynamic analysis should be undertaken.

9 Conclusion

The use of Rail Track-Structure Interaction analysis software has many benefits over manual methods. Automated model building guarantees correctly-built models compared to manual model creation that may require extensive checking along with the project time savings associated with this. The material properties associated with the track/structure interface are automatically updated according to the position of the passing train or trains for all analyses.

Results are automatically provided in summary, tabular or graphical formats for all or selected parts of the track/bridge model. Overall it provides a faster assessment of thermal and / or train loading track interaction effects on multi-span structures to the UIC774-3 Code of Practice than all other known methods.
Eurocode EN 1991-2:2003 Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges encompasses significant elements of the UIC 774-3 modelling approach when evaluating the combined response of the structure and track to variable actions. For a UIC60 rail, the limiting design criteria are the same as those specified in the International Union of Railways Code UIC 774-3 meaning that Rail Track-Structure Interaction analysis software can be directly employed to meet Eurocode requirements.

References

[1] UIC Code 774-3 R. Track/bridge Interaction. Recommendations for Calculations (2001) Union Internationale des Chemins de fer, Paris, France.

[2] BS EN 1991-2:2003 Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges. BSI, May 2008

[3] LUSAS Finite Element Analysis Software, LUSAS, Kingston upon Thames, UK

[4] Lee, Joo-Heon et al.: Track–Structure Interaction analysis and application on Kyoung-bu 2nd stage HSR Project, Journal of the Korean Railway Society, May 2006, pp. 7-13.

VIBRATION PROBLEMS AT SWITCHES

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Abstract

In principle, any rolling load generates vibrations. When passing switches, there is an increase of vibration emission caused by railway traffic. Especially at high speeds and high axle loads, significant vertical forces occur due to dynamic loads at the tips of rigid frogs. Additionally, wheels striking the check rails induce jerky horizontal forces. In switches with movable frogs, compared to types with rigid frogs, the gap at the frog is closed. On the basis of numerous measurements on simple switches with rigid or movable frogs, the differences of the vibrations are investigated and compared with literature references. The measurements of the selected switches show that, especially within the frequency range of 30 Hz and poor track quality, trains passing a switch generate a significant increase of vibration emissions. Furthermore, the vibration gain between switch and open track decreases with the distance from the switch to the observer. In one measurement, all regional trains passed a switch with rigid frog with changing directions. Independent from the direction of travelling, these regional trains caused similar vibrations. Movable frogs, however, are suitable for vibration reduction: They amplify the vibration emissions less than switches with rigid frogs.

Keywords: vibration, switch, rigid frog, movable frog, vibration gain

1 Introduction

In civil engineering, vibrations and, in connection, protection against them, have been playing an increasingly important role over the last years. The increasing occurrence of noise immission reducing facilities (e. g. noise barriers or noise control windows) leads to a stronger perception of vibration immissions.
In principle, any rolling load generates vibrations. Vibrations are introduced into the ground and following spread out as vibrations. With the vibration propagation, the frequencies are altered and the vibrations are damped additionally. The vibration excitation including radiation is called emission and the vibration propagation is called transmission. Foundations of buildings are excited to oscillate by vibrations. The vibrations are generally significantly reduced by the effect of coupling. Via the foundations, the walls and ceilings are also activated to oscillate. The self-oscillation behavior leads to a significant gain of vibrations at ceilings. The entry of the vibrations into the structure of buildings and their perception by people is called immission. Fig. 1 shows the variation of vibration propagation due to rail traffic and the decrease of vibration velocity with increasing distance.
In the ideal half-space, two types of waves occur by an excitation at the surface. On one hand there are body waves, which spherically propagate in three dimensions, and on the other hand there are surface waves that mainly propagate horizontally with limited depth. Body waves are divided into primary waves and secondary waves, surface waves are divided into Rayleigh waves and Love waves. Primary waves oscillate in propagation direction. Secondary waves and Love waves oscillate transversely to the propagation direction. Rayleigh waves

include both longitudinal and transverse motions. Rayleigh waves, which are very similar to water waves, assume the greatest part of the energy transfer for the vibration propagation in the ground, caused by rail traffic.

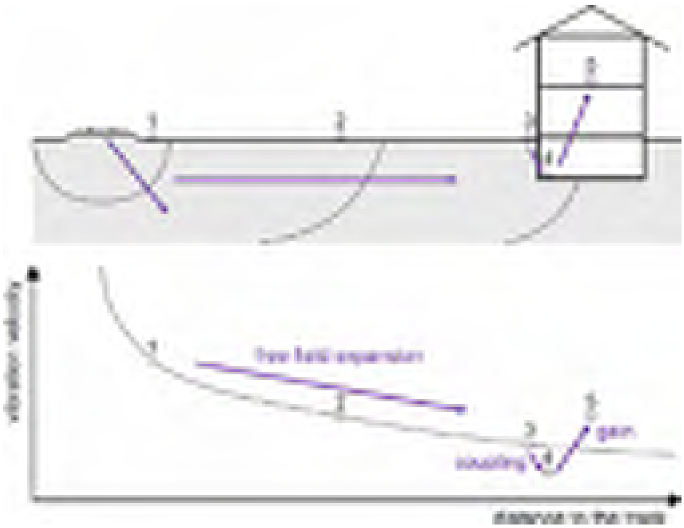

Figure 1 Vibration propagation

Trains in motion generally resemble a line source. When passing switches, there is an increase of vibration emission caused by railway traffic. Especially at high speeds and high axle loads, significant vertical forces occur due to dynamic loads at the tips of rigid frogs. Additionally, wheels striking the check rails induce jerky horizontal forces. These vertical and horizontal forces are similar to a point source.

Vibrations are damped during their propagation. A differentiation can be made between material damping and geometrical damping. The energy loss in the material damping is caused by internal friction. The reduction of the geometric damping is caused by the fact that the introduced energy is distributed across an increasingly larger area. Table 1 shows the propagation in the ideal half-space for geometric damping. The greatest reduction occurs by body waves at point sources, while surface waves at line sources move unchanged.

Table 1 Propagation in the ideal half-space for geometric damping [1]

Type of wave	Reduction for point sources	Reduction for line sources
Body wave at the surface	$v = v_0 * (\frac{r_0}{r})^2$	$v = v_0 * (\frac{r_0}{r})^1$
Surface wave	$v = v_0 * (\frac{r_0}{r})^{0,5}$	$v = v_0 * (\frac{r_0}{r})^0$

where:

v vibration velocity in distance to the source;
v_0 vibration velocity in reference distance to the source;
r_0 reference distance to the source;
r distance to the source;
exp. damping exponent for geometric damping.

2 Measurement of vibrations

2.1 Measuring system

For vibration measurements, the measurement instrument MR 2002 by Syscom was used. Frequencies between 1 and 315 Hz with amplitudes between 0.0001 mm/s and 115 mm/s could be measured.

2.2 Measurement points

For the location of measurement points, special emphasis was put on the following points: Switches in the area of flat and open terrain were selected. Embankments and incisions, buildings or other barriers would affect the results and the measurements would be difficult to compare. Furthermore, it had to be considered that only one simple switch is located in the area of the chosen measurement. Switches in the immediate vicinity would affect the results of the measurements.

Only double track lines were selected. The gain factor is calculated by the vibrations from the track with a switch divided by the track without a switch. In addition, it was important that there are no joint clearances in the area of the measurement points. Joint clearances would also affect the results.

2.3 Vibration analysis

For the analysis, only trains in similar velocity ranges were evaluated. For this reason speed normalizations could be avoided. Fig. 2 shows a typical time signal without the influence of a switch. Fig. 3 shows a signal with the influence of a switch. The vibration velocity is plotted in mm/s on the vertical axis and the horizontal axis represents the time in seconds.

Figure 2 Typical time signal without the influence of a switch

Figure 3 Typical time signal with the influence of a switch

3 Results of measurements

3.1 Results of rigid frogs

In one measurement, all regional trains passed a switch with rigid frog with changing directions. Therefore, a comparison of the vibrations in relation to the direction of travel was possible. Fig. 4 indicates the functions of regional trains in movement direction towards the start of the switch or towards the end of the switch. These regional trains caused similar vibrations independent from the direction of travelling. Fig. 5 shows a summary of all results for rigid frogs, divided into sections ‹ 25 m, 25 m – 40 m and › 40 m distance at right angles to the track axis.

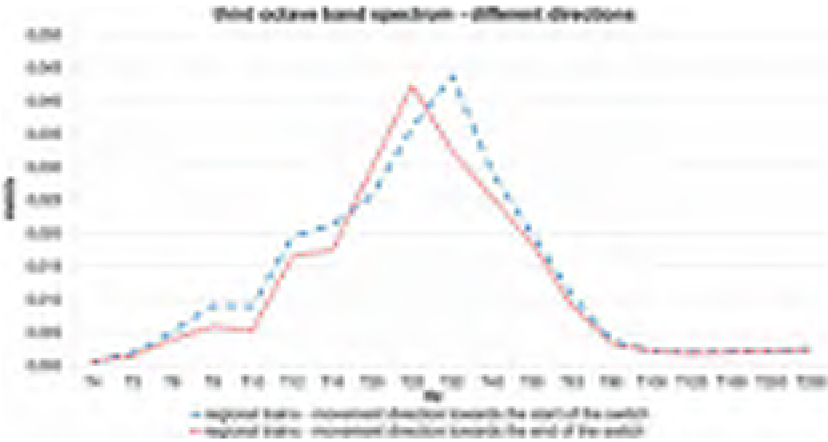

Figure 4 Third octave band spectrum – different directions

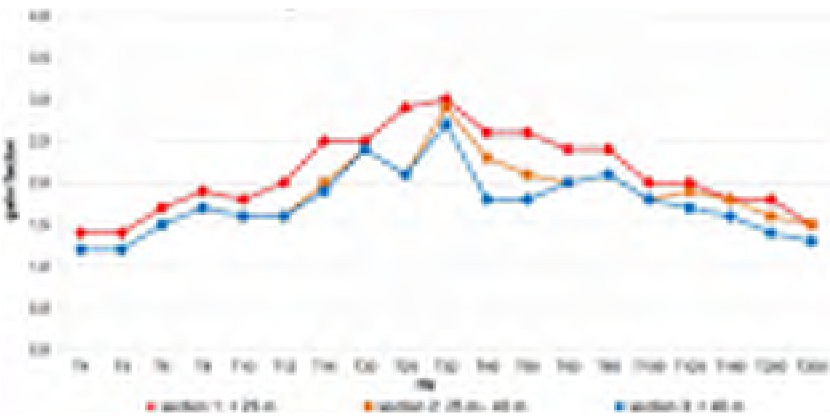

Figure 5 Gain functions – different sections

For the generation of the gain function for the section ‹ 25 m, results of measurements series with good and poor track quality were available. The gain function for this range agrees well with the functions from the literature (see Section 4). The unexpectedly large gains and the small decrease in gain with increasing distance for the sections ≥ 25 m can be traced back to the poor track quality in the area of these measurements. The measurements of the selected switches show that, especially within the frequency range of 30 Hz and poor track quality, trains passing a switch generate a significant increase of vibration emissions.

3.2 Results of movable frogs

For the measurements with movable frogs, the distance from the switch to the measurement instrument was selected with approximately 16 m. The investigated switches have a spring-mounted sideways movable frog tip. In one measurement, long-distance trains drove across a switch with a movable frog with different speeds. The rapid trains drove at speeds of approximately 180 km/h and the slower ones passed at approximately 90 km/h. Fig. 6 shows a comparison of the gain function of these long-distance trains. Interestingly, for fast trains the gain function is lower than for slow trains in several third octave band frequencies between 6 Hz and 125 Hz. Only the range of 10 Hz, 12 Hz, 50 Hz and 125 Hz show higher gains for fast trains.

Figure 6 Gain functions – different velocities

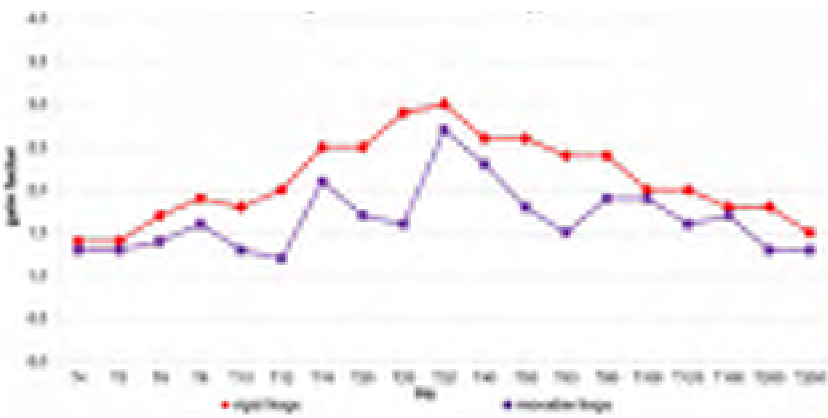

Figure 7 Gain functions – different types

Fig. 7 shows a comparison of the gain function of rigid frogs with movable frogs for distances of approximately 16 m. The gain function with movable frogs is lower than that with rigid frogs in the entire relevant frequency range, with the largest improvements in the third octave band frequencies of 12 Hz, 25 Hz and 63 Hz. According to the analysis of this work, movable frogs are suitable for vibration reduction. They amplify the vibration emissions less than switches with rigid frogs.

4 Comparison with literature

This chapter provides information on data in the literature of the influence of switches. It should be noted that the data in the literature varies significantly. Furthermore, the influence of switches is sometimes expressed in decibels and not in factors. With Eq. (1) one can convert decibels into factors.

$$a = b^{\frac{x}{20}} \quad (1)$$

where:

a factor;
b base (value 10 for sound);
x decibels.

Fig. 8 shows several functions from the literature for switches with rigid frogs and distances ‹ 25 m. For comparison, the red function with rhombuses from Fig. 5 is also shown.

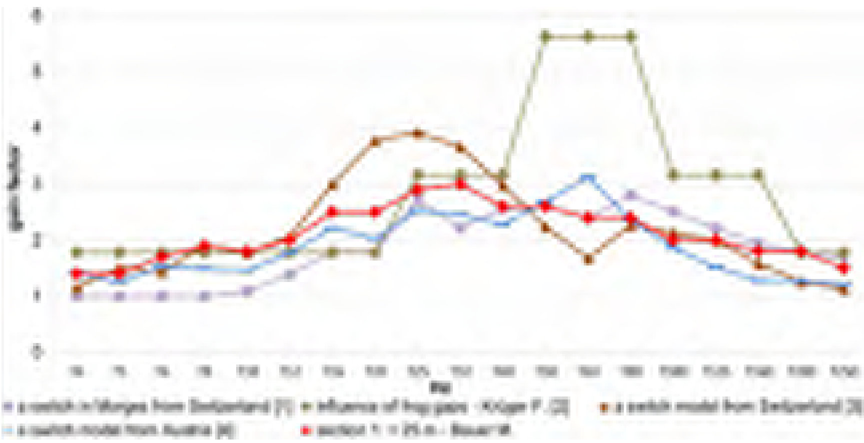

Figure 8 Gain functions – section 1: ‹ 25 m – comparison with literature

The violet function with squares shows the influence of a switch in Morges, Switzerland [1]. This gain function is usually lower than the red one of this investigation. The low factors may be associated with the fact that a short period between measurement and installation of the switch is available and the switch therefore has a very good condition.
The green function with circles describes the influence of frog gaps [2]. The gains are in the upper frequency range above the defined function of this work. The maximum gain occurs at a factor of 5.6 in the range of 60 Hz. High frequencies are rapidly attenuated with increasing distance. Therefore, it can be assumed that this gain function applies directly to the area of the wheel-rail-interaction or, for short distances, to the gap of the switch.
The gain function of a switch model from Switzerland applies to distances of 8 m [3]. This brown function with triangles results from numerous measurements. The results of these individual measurements vary greatly. This brown function is in numerous frequency ranges high which can be attributed to the short distance between switch and measurement instrument. The blue function with straight lines shows a switch model from Austria [4]. In addition to numerous measurements from switches with rigid frogs, a switch on a slab track was also taken into account for the determination of this function. In a large frequency range, this gain function is lower than the function of this investigation. In the third octave band of 63 Hz the factor is well above the factor for section 1.

5 Conclusion

The measurements of the selected switches show that, especially within the frequency range of 30 Hz and poor track quality, trains passing a switch generate a significant increase of vibration emissions. Movable frogs, however, are suitable for vibration reduction: They amplify the vibration emissions less than switches with rigid frogs.
The investigations carried out allow for a more precise model for vibration predictions with respect to the distance from the affected object and also from the frequency dependence of the vibration gain.

References

[1] Ziegler Consultants – Erschütterungsausbreitung beim Schienenverkehr, http://www.sga-ssa.ch/pdf/events/Ziegler_Erschuetterung_Eisenbahn_Text.pdf, 02.01.2014.

[2] Krüger, F.: Begriffe, Normen und Datensammlung, Schall- und Erschütterungsschutz im Schienenverkehr, second edition, Expert Verlag, Renningen, pp. 498-507, 2006.

[3] Müller, R.: Schwingungsreduktion bei Eisenbahnanlagen, VDI Berichte, Baudynamik Tagung, Kassel, pp. 483-496, 2006.

[4] Tappauf, B.: Erschütterungs- und Körperschallprognose für den „Semmering Basistunnel neu“, Bauwerksdynamik und Erschütterungsmessungen, 13. Symposium, Dübendorf, 2010.

MEASUREMENT AND ANALYSIS OF THE DYNAMIC EFFECTS ON THE CROSSINGS

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Abstract

The turnouts are one of the key points of the railway routes. In particular, turnouts in the main tracks are passed relatively high speed. This paper is focused on measurement and analysis of dynamic effects on railway turnouts. The main rail turnouts which are passed in speed of 160 km/h^{-1} and with different wear condition were selected. Attention was focused mainly on the crossing panel of the turnout, where the highest dynamic impacts occur. The turnout was measured twice: before and after frog repair by welding. The point of the paper is comparison of the crossing part of turnout in term of dynamic behaviour and assessment of influence of the transition geometry (wearing depth) on dynamic effects. Because dynamic impact from the wheel sets during the transition from the wing rail to the nose of the crossing depending on the quality of the transition geometry and also on the overall stiffness of the rail superstructure, the appropriate methodology of the measurement was designed. The methodology is designed for in situ measurement in condition of full operation and consists of two parts. Measuring of vibration and measuring of shifts of the bearers in the crossing panel. Piezoelectric vibration acceleration sensors (the accelerometers) were attached to plastic handles and glued to the cleaned surface of the measured structure. Three-axis accelerometer B&K 4524 B001 was placed at the foot of the wing rail, five accelerometers B&K 4507 B001 were placed at the bearer and one accelerometer B&K 4507 B004 was placed at the measuring bar embedded in the ballast. Shifts of the bearers ware measured by inductive displacement sensors HBM WA-10 T which were attached through magnetic holders to special frames. Frames are made of two steel bars embedded in the structural layers of subgrade and steel cross members connecting the two poles. This method created referential zero-point shift. For evaluation of measured data were used time analysis, frequency analysis using the amplitude spectrum and time-frequency analysis.

Keywords: turnout, crossing panel, vibration measurement, measuring of shifts, frequency analysis, time-frequency analysis

1 Introduction

The switches and crossings are the main parts of railway lines. Although the length of tracks with switches and crossings is relatively short, the maintenance of switches and crossings are approximately same expensive as maintenance of rest of the plain line. It is due to complicated force action when a train is passing the crossing and as well due to the maintenance of many different parts which crossing is built of. Except that the maintenance of the crossings carries expensive direct costs, it also carries non direct costs when repaired, for example: delay of trains, costs for alternative transport and so on. It's very important to plan the regular

maintenance of these parts very carefully. If regular maintenance is not done in time, the price for immediate repair is increasing very fast.
There is no complex methodics for measuring and evaluation of the dynamic effects on railway crossing in Czech Republic so far. Authors have been continuously working on solving this issue. They study development of the measuring methods and analysis of the dynamic effects as well, so it would be possible to plan repairs and the maintenance of crossings on time and with minimum costs. In this paper we are concerned with comparison of the dynamic behaviour of the railway crossing before and after repair of the frog by welding.

2 Description of the problem

Continuous demand on the bearing capacity leads to higher rigidity of construction layers and subgrade. The concrete sleepers are mainly used nowadays. Compare to wood sleepers their bending stiffness is much higher but on the other hand they are less plastic. All these aspects lead to higher stresses in the ballast, which change its shape under increased load and thus change geometric parameters of the track. Uneven support of the sleeper is then created and dynamic effects are increased which leads to faster degradation of the ballast. In the crossings, mentioned problems are combined with change in stiffness of the track and dynamic action when the wheels are passing from the wing rail to the crossing nose or vice versa.
Combination of these factors cause the fact that the most stressed point of crossing construction is the frog part where the wheel is crossing from the wing rail to the crossing nose. On this place the dynamic impact occurs and its magnitude is influenced by quality of the transition geometry (from the wing rail to the crossing nose). This impact is carried through the sleeper to the ballast, which is for this reason extremely stressed. This extensive load causes abrasion between gravel and sleeper. Whole process results in degradation of shape of the ballast under the sleeper, which leads to insufficient support of the crossing. If the crossing is not properly supported it collapses transition geometry and degradation process speeds up.

2.1 Possible solution of the problem in practice

Planning of the maintenance and repairs of crossings in Czech Republic is mainly based on the visual control and basic geometric measurements directly in situ. Most of the time there is no defect prevention and later it's necessary to deal with damage which is more complicated and more expensive as well. Repair of damage is mostly done by frog welding, underlaying base plates and tamping of construction. Unfortunately these repairs have only short time effect and defects are detected again, sometimes developing even faster.
The aim of author's team is to complete present system of geometric observation and wear measurement for new element of monitoring which would be measurement of dynamic effects. Thanks to this new approach it would be possible to evaluate influence of repair works on dynamic effects. With long-term measurement of dynamic effects it would be possible to schedule regular maintenance better and prevent defects. This could not only save financial funds but also increase safety and fluency of the railway traffic.

3 Methodology of the measurement

The methodology of the measurement has two parts. The first part is focused on dynamic behaviour of construction and follows up transmission of the vibration from rail to sleeper and to ballast. The second part is designed to follow up construction movement under the load.

3.1 Vibration spreading in the crossings

Probably the most interesting thing from point of analysis and evaluation is following up the vibration acceleration transmission from the frog through the sleeper to the ballast when vibration is caused by the wheel sets. It's considered that small part of vibration is absorbed by the rail (frog) and rail pad, another part of the vibration is absorbed by the bearer and the rest of the vibration is transmitted to the ballast. Authors use piezoelectric vibration acceleration sensors (designation A) for measuring values of the dynamic effects.

The sensors are placed in such a manner that it's possible to follow up spread of vibration energy through the whole system. Three-axis accelerometer placed on flange of the wing rail is detecting magnitude of dynamic impact, which is placed on frog (A4Z, A5X, A6Y). Vertical dynamic impact posed on frog (A4Z) is detected in vertical direction, respectively the part which is transmitted to flange of the wing rail. The other sensors are one-axis sensors and are placed on such positions to best detect spreading of dynamic impact from the frog through the sleeper and into the ballast. Position of the sensors is as follow: A3Z – on bearer nearest to the crossing nose; A0Z – on the measure bar embedded to the ballast close to crossing nose. The vibration transmissions through the bearer under crossing nose: A1Z – on head in straight direction; A2Z – on line of the track in straight direction; A3Z – on bearer nearest to the crossing nose; A7Z – on line of the track in curve direction; A8Z – on head in curve direction. Thanks to the described methodology of the measurement it's possible to obtain complex information about how dynamic impacts are transfer in close proximity of the frog and into the ballast.

3.2 Measuring of moving behaviour of the construction

Moving behaviour in place of the frog is followed up by the induction sensors (designation S) which measure vertical displacement of the bearers.

The sensors S0, S1, S4, S7 are placed on single bearers along the crossing as close as possible to wing rail. Measurements from these sensors help us to imagine bending curve which occurs along the crossing. Maximum displacement is expected to be directly under the crossing nose.

Vertical displacements of the most loaded bearer (most likely under the crossing nose) are measured by following sensors: S2, S6 – placed on the head; S3 – placed in the line of the track in straight direction; S4 – placed nearest to the crossing nose; S5 – placed in the line of the track in curve direction.

The sensors of vertical displacements (S0, S1, S2, S3, S4, S5, S6, S7) are attached to steel beam with special magnetic holder. The frame consists of two steel rods with diameter 20 mm and length 70 cm. The rods are hammered to the ballast to depth of 65 cm. The axial distance from one another is 60 cm. The steel beam is attached to steel rods. The length of steel beam is 70 cm.

If trains are passing mostly through straight direction and curve direction is not that busy it could happen that bearers are lifting upward when a train is crossing. Thanks to position of the sensors these changes could be detected.

4 Mathematical apparatus

When dynamic action from railway traffic is measured in-situ it's necessary to describe and evaluate stochastic signal. This is rather difficult. In order to get necessary information and compare the dynamic action on each construction we can evaluate stochastic signal from three different perspectives. First perspective is time scale. We will calculate surface under the curve of moving RMS (RMS stands for Root Mean Square). Thanks to this time perspective it's possible to determine total dynamic action on construction. If we compare area under

curve of moving RMS with area under curve of moving maximum amplitude we get a value called Crest Factor. Crest Factor represents dynamics of the signals.
For detailed analysis of the dynamic actions above described method is not sufficient because frequency composition is not known. For this reason it's useful to transform signal from time scale to frequency scale. For this transformation we use Fourier transformation method. In frequency scale it's possible to determine which parts of signal are the strongest ones. For detailed analysis it's useful to use third perspective, which evaluate signal from time-frequency scale [3]. With this perspective we can see not only frequency composition but also its occurrence in time.

4.1 The Welch method

The Welch method is certain modification of the Fast Fourier Transformation. Digital signal x[n] (n=0,1,2,..., N-1) is divided into K segments each of them with length M (xi[m], i=0,1,..., k-1, m=0,1,...,M-1). Segments are either placed in row one by one then N=K·M or they overlap. Each segment is weight by function w[m]. After transformation and following calculations periodograms components Sj[k] are created. These components put together represent approximate spectral density S[k]. This estimation is described by following formulas. Component of periodograms is determined by formula [1].

$$S_j[k] = \frac{1}{U \cdot M} \cdot \left| \sum_{m=0}^{M-1} x[m + i \cdot M] \cdot w[m] \cdot e^{\left(\frac{-j2\pi mk}{M}\right)} \right|^2 \tag{1}$$

where:

$$U = \frac{1}{M} \cdot \sum_{m=0}^{M-1} w^2[m] \tag{2}$$

is vector's standard of window function, w[m] is window function. Resultant estimation is done by averaged component periodogram.

$$\hat{S} = \frac{1}{K} \cdot \sum_{i=0}^{K-1} S_j[k] \tag{3}$$

4.2 Short Time Furier Transform (STFT)

STFT provides compromise between time and frequency signal representation [2]. Its integral definition is:

$$STFT_x^{(\omega)}(t\ ,f) = \int_{-\infty}^{\infty} \left[x(t) \cdot g^*(t - t\ ) \right] \cdot e^{-i2\pi f(t-t\ )} \cdot dt \tag{4}$$

where:

g	window function;
'*'	complex conjunction;
t'	time displacement of window;
x(t)	time representation of signal;
$STFT_x^{(\omega)}(t',f)$	time-frequency representation [2].

5 Description of measuring location and measured crossing

The chosen crossing was the fix common crossing of turnout number 59 in station head of the railway station Chocen. The turnout track system: rails 60E1 on concrete bearers, fastening system Vossloh SKL 24 and ballast. Trains run in trailing direction. Turnout crossing angle is 1:14 and radius 760 m. The crossing of the turnout was measured twice: the first time before repair and the second time after repair of frog by welding, half a year later with previously mentioned methodics. The following measurements have been completed in full operation: construction moving behaviour, acceleration of vibration, geometry of transition by very accurate levelling and measuring with laser device for the measurement of cross sections of frogs.

6 Evaluation of measured data

Due to scope of our paper we are only interested in transmission of vibration from wing rail (A4Z) through bearer (A3Z) to the ballast (A0Z) and moving behaviour of bearers along the frog. Equally 20 trains were measured before and after repair of frog. From many graphs and tables we choose those which best represent results of our analysis.

6.1 Evaluation of vertical displacement of sleepers

From each train maximal vertical displacements were measured. Maximal vertical displacements were used to create bending curves along frog. Then, from all measured trains the envelope curve of maximal and minimal vertical displacements was created and finally the average displacement curve was created as well. This analysis was done before and after repair. Sensor S0 was placed two bearers in front of the crossing nose of the frog, sensor S1 was placed one bearer in front of the crossing nose, sensor S4 under crossing nose and sensor S7 one bearer behind the crossing nose. The figure 1 shows the fact that vertical displacements after repair are greater than displacement before repair. It's obvious that the repair doesn't stop the development of the vertical displacements. Dash line on figure 1 represent displacement before repair and full line represent displacement after repair. It's interesting that the maximal displacement is not under crossing nose of the frog as it was in previous analyses we have conducted on other crossings. It's actually shifted to the next bearer.

6.2 Evaluation of transmission of vibrations in construction

The evaluation of acceleration of the vibration is divided into three different perspectives. First perspective is time area, where we represent evaluation with Crest Factor, which is ratio between maximal amplitude and RMS. This factor is calculated as an area under curve of moving maximum amplitude and moving RMS which are compared. The table 1 represents the measurements from selected trains and their comparison before and after repair. The table 1 shows the fact that dynamics of the signals after repair are smaller than before repair. In other words we can see positive influence of repair on dynamic effects.
The second perspective is frequency area. Welch Method was used for all trains going over 120 km/h. All graph results were put together and one average graph for each sensor was created. For each sensor we get average graph before and after repair. Figure 2 shows that from frequency point of view repaired crossing is slightly worse than unrepaired one. Decline in vibration was detected only for 50 Hz frequency; the other frequencies show worse or equal results after repair. Significantly worse values after repair were measured for frequencies 75 – 150 Hz on bearer (A3Z) and for 150 – 700 Hz on wing rail (A4Z). In figure 2, dashed line is used for frequency measured after repair and solid line is used for non-repaired crossing.

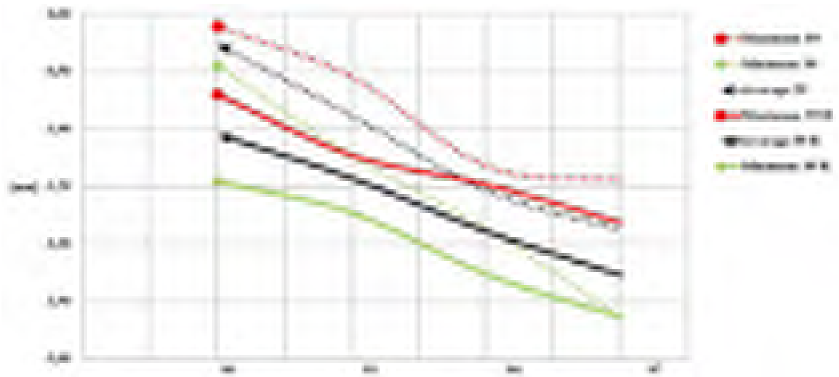

Figure 1 Vertical displacement curves along the crossing

Table 1 Evaluation with the Crest Factor

Train	Crossing 59				Crossing 59 repaired			
	Speed [km/h]	AOZ	A3Z	A4Z	Speed [km/h]	AOZ	A3Z	A4Z
Leo Express	157	4,38	2,78	2,81	156	4,64	3,31	3,09
163	49	4,34	3,52	8,48	51	2,53	2,09	3,07
RegioJet 363	137	5,57	3,67	4,21	139	5,09	3,09	3,10
Taurus	141	4,28	2,92	2,95	140	5,28	3,14	3,20
Pendolino 680	155	4,61	2,90	2,95	163	5,24	3,51	3,12
471 City Elefant	75	3,31	2,11	2,76	72	2,50	2,03	3,00
151	125	3,65	2,70	2,87	120	4,57	2,79	3,12

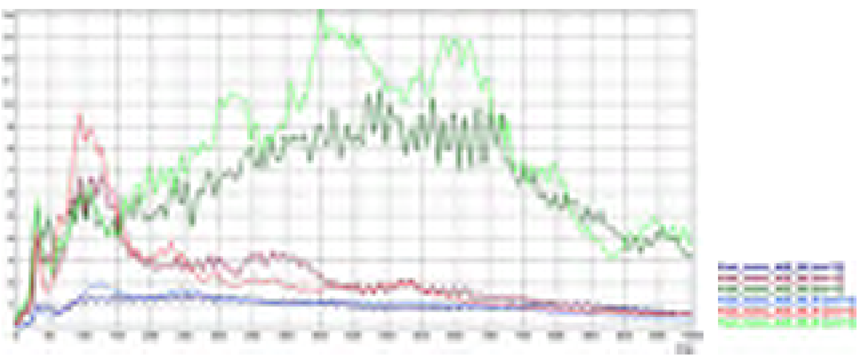

Figure 2 The Welch method analysis

Time-frequency method can be used for confirmation of conclusions from frequency and time analysis. The advantage of this method is that it can display time and value of frequency action in one chart. As an example we present time-frequency graph calculated for signal detected on wing rail when Pendolino train passed the crossing. As we can see on figure 3 on horizontal axis is time and on vertical axis is frequency. The figure 3 shows the fact that the densest frequencies were detected in time when train wheel sets were passing measuring point. Maximum frequency range is from 400 – 550 Hz and 600 – 750 Hz, which correspond to the frequency analysis.

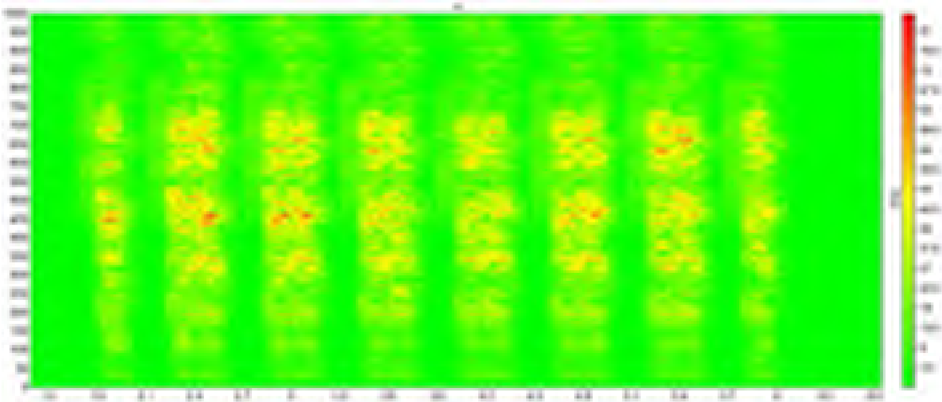

Figure 3 Time-frequency analysis with method STFT

7 Conclusion and recommendation

From conducted measurements is obvious that it's not efficient to repair the frog only by welding because dynamic actions are increasing even after repair. Detected signal magnitude degrees after repair (which is visible on Crest Factor) but overall dynamic actions are increasing even after repair. This conclusion is supported by practical experience as well, where repair effect doesn't last for a long time and defects are detected again. In this particular case it could be problem with track bed because vertical displacements under bearers are increasing. Ideal solution would be stabilization of track bed with combination of the ballast tamping and frog welding.

References

[1] Smutný, J.: The modern method of the noise and vibration analysis applied on railway traffic, Theses of the doctoral dissertation, VUT Brno, ISBN 80-214-0988-6

[2] Smutný, J., Pazdera, L.: Time-frequency analyses of real signals, 2003, ISBN 80-86433-23-4

[3] Smutný, J., Pazdera, L., Bajer, M., Moll, I.: The Analysis of Dynamic Effects in Turnout Structures by the Method of Quadratic Time and Frequency Invariant Transformations, Proceedings f the Sixth International Symposium "Computatuonal Civil Engineering 2008" Iaşi, ISBN 978-973-8955-41-7, Societatea Academica Matei – Teiu Botez, Iasi, Romania, 2008

ADVANTAGES OF INSTALLATION OF RUBBER-METAL ELEMENTS IN SUSPENSION OF RAILWAY VEHICLES

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Abstract

During the exploitation of railway vehicles, failures of elements of the suspension are very often. The fractures on the components of suspension are especially characteristics for freight wagons operating in extreme loading conditions. This causes a decreasing of the efficiency of rail transport and very often can cause a derailments with huge material damage and human victims. The task of this paper is to explore the possibilities for improvement of suspension behaviour through subsequent installation of rubber-metal elements of simple design. The methodology is based on the identification of causes of failures through numerical and experimental analysis of suspension behaviour. These research is basis for design of simple solutions of rubber-metal elements which can be subsequently installed in suspension of wagons. In this paper the proposed methodology is applied on the freight wagons for coal transportation whose suspension is based on laminated springs. Subsequent installation of rubber-metal elements is resulted with prevention of very frequent failures on elements of suspension. Besides, this solution enables reducing of maintenance costs, increasing the efficiency of transportation, and enables many other advantages.

Keywords: rubber, suspension, railway vehicles, wagons, laminated spring

1 Introduction

One of the most important parameters which determine the reliability and running safety of railway vehicles is functionality of the suspension. The malfunction of suspension causes very serious consequences and in many cases may cause derailment. For this reason, the design and reliability of suspension of railway vehicles has previously been the subject of many papers, including those concerning the detection of faults and analysis of failures [1–4]. The aim of all these researches was to indicate the potential problems and to give the motivation for improvements in existing or newly-designed solutions of suspension. Many studies show that failures of elements of the suspension are particularly frequent when the wagons are used in extreme operating conditions. One such system is, for example, the system of railway transportation of coal from mining basin ''Kolubara'' to the thermal power plant ''Nikola Tesla'' Obrenovac, Serbia. The transportation is based on the Fbd wagons and this line is among the most busiest industry railway lines in Europe. Because of this and because of some specifics in the design of suspension which is based on the laminated springs, the very frequent fractures of spring leafs were caused derailments in many cases (one such case is shown in Fig. 1). The consequences of derailments were huge material damage and significant decreasing the efficiency of railway transportation. Such problems are very often on many loaded railway lines for the transport of cargo. In these lines the suspension of freight wagons is usually based on the laminated springs. In the case of such problems, the logical way is to explore the failure of laminated springs and to improve the suspension system. The main task of improving of suspension is to be economical and to allow quick implementation. The one of the main idea for solution the problem is subsequent installation the rubber-metal element in suspension. The motivation lie in the fact that rubber can significantly improve the behavior of suspension and dynamic characteristics of railway vehicles. This is confirmed by numerous studies [5–9].

Figure 1 The derailment caused by the failure of the suspension system

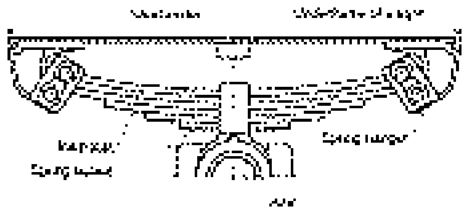

Figure 2 The scheme of suspension of Fbd wagon

2 Analysis of failure of suspension

The typical scheme of suspension of freight wagon that is based on the laminated springs is similar to those for Fbd wagon shown in Fig. 2. The steel limiter is fixed for the underframe of wagon and has the task to limit the stroke of laminated spring. In extreme operating conditions at maximum loads there are intense dynamic rigid impacts of spring buckle in the steel limiter which is very unfavorable for the suspension and the underframe of wagon.

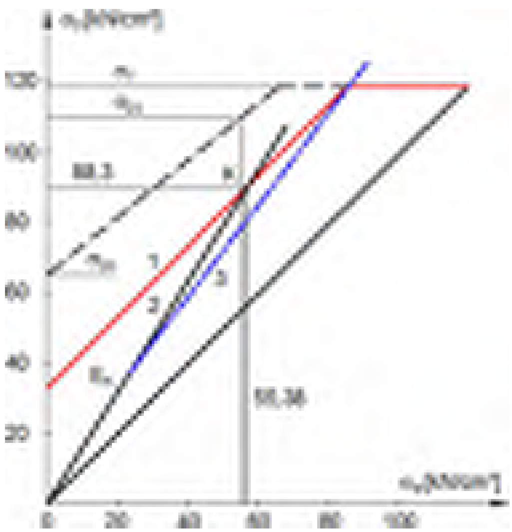

Figure 3 The Goodman-Smith diagram

In the case of Fbd wagons used in the thermal power plant "Nikola Tesla", statistical analysis has shown that at the annual level, per one wagon there are almost 3 fractures on the elements of the suspension system. In addition, it is found that the most dominant are the fractures of the laminated springs. The start of solving the problem should be experimental testings of laminated springs with the aim to analyzing and discovering the causes of failure of suspension.

In the case of Fbd wagon, during the experimental tests, the behavior of laminated spring in the exploitation was recorded. The recorded data of vertical deflection of the spring buckle from exploitation were used to form a Goodman-Smith diagram and determine the lines of operating and the critical stresses of laminated spring (Fig. 3). The methodology with more details is shown in paper [10].

The spectrum of force amplitude or stress of laminated spring corresponds to the hard working regime. Based on the characteristics of laminated spring material the line of main dynamic strength was formed (dashed line), where the characteristic values are given in Table 1.

Table 1 The characteristic values of Goodman-Smith diagram

Yield strength	$\sigma_T = 110 \div 125$ kN/cm^2
Dynamic strength during the alternating variable load	$\sigma_{Dn} = 60 \div 70$ kN/cm^2
Dynamic strength during the DC variable load	$\sigma_{Dj} = 110$ kN/cm^2

The extreme values of these data (σ_T, σ_{Dn}) have low probability of occurrence, so in the further analysis the mean value of given areas are used. The critical stress of laminated spring (red line) was obtained by correction of main dynamic strength by the factor k_A. This factor takes into account the quality of the laminated spring production, the conditions of exploitation, and uniformity of loading are random variables. On the other hand, loads during the exploitation of laminated spring caused the stresses shown in Table 2 [11].

Table 2 The characteristic values of Goodman-Smith diagram

Medium dynamic stress in the laminated spring	$\sigma_{sr} = 55.38$ kN/cm^2
Maximal dynamic stress in the laminated spring	$\sigma_{max} = 88.3$ kN/cm^2

Change of the operating stress of the laminated spring is linear and in the Goodman-Smith diagram it is represented by the line 2 which passes through the origin and the point K which has coordinates (σ_{sr}, σ_{max}). In this case the line of operating stress of laminated spring cuts the line of critical stress.

From analysis of diagram it is concluded that in the existing state of laminated spring the occurrence of fracture is very likely. It is also concluded that occurrence of maximal stresses mostly affected the fractures of main leafs of laminated springs.

Therefore, the main reasons for the formation of fractures were primarily increased stresses and loads, and unreliable quality of laminated spring production. Increased loads arising not only due to overload of wagons, but also because of their uneven loading that cannot be accurately controlled.

3 Improvement of suspension

Based on previous considerations and with respecting the existing design of the wagon, the special rubber-metal element for subsequent instalation in the suspension can be designed. The design of rubber-metal element for Fbd wagon is shown in Fig. 4.

In this case, it is predicted that the life time of rubber-element must be minimal 5 years. This should enable the element to be replaced in frame of regular servicing of Fbd wagons. The element is very easy to install in existing wagons, between the laminated spring buckle and underframe, as shown in Fig. 5.

Figure 4 The rubber-metal elastic element

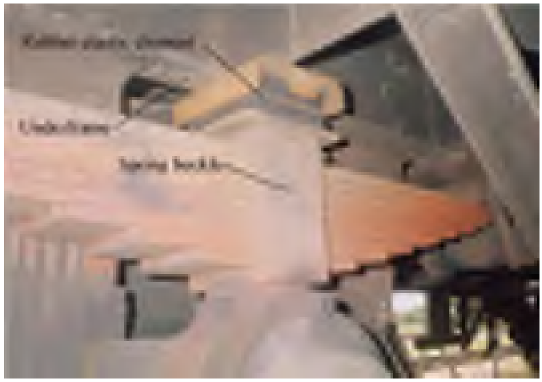

Figure 5 The rubber-metal elastic element instaled in suspension

In order to determine the optimal characteristics of rubber, several samples of different manufacturers and stiffness are tested in exploitation conditions. The main aim was to find compromise between the laminated spring relieving, the life-time of rubber-metal element, and dynamic characteristics of the whole wagon (number of occurrences and the values of the stress amplitudes – deflection as a function of the path traveled).
For the purpose of registering the mentioned dynamic sizes a special measuring system was designed. This measuring system is placed in the wagon and is based on the counter with the task to register the appearances and values of the amplitude of the laminated spring in a longer period of time during the exploitation of wagon. Analysis of results is shown that best solution is with rubber stiffness of 42 Shore. The diagram of optimal zone of stiffness formed on the basis of available space and deflection limitations is shown in Fig. 6. The diagram of stiffness of finaly adopted rubber-metal element obtained by laboratory experimental tests is shown in Fig. 7.

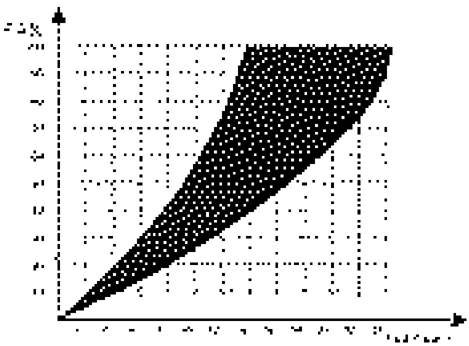

Figure 6 The zone of optimal stiffness

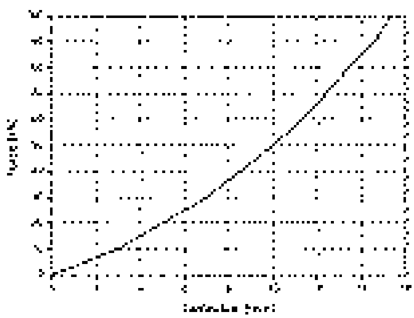

Figure 7 The diagram of stiffness of finaly adopted rubber-metal element

Subsequent installation of designed rubber-metal element in the suspension should cause significant decrease of the stress amplitude, and therefore a new line of operating stress of laminated spring in Goodman-Smith diagram (blue line 3). This should result in significantly lower stress amplitudes, and hence sholuld reduces the extent of fatigue loading. Therefore, the blue line 3 on the Goodman-Smith diagram represents the compromise between the laminated spring relieving and load of rubber-metal element, which provide a permanent dynamic strength of the laminated spring.

The designed rubber-metal elements are installed into the suspension of one Fbd wagon on which the experimental tests in exploitation were performed. The results of these tests are shown in the next chapter.

3.1 FEM analysis of rubber-metal element

The method of finite element (FEM) has been used to analyze the stress distributions and deflection of the rubber-metal element. The nominal loading range used is 100 kN. The calculated deflection of rubber-metal element is shown in Fig. 8. The obtained results have shown that rubber-metal element satisfy the stress limitations. Also, the obtained deflections is matching with those obtained by the laboratory experimental tests (Fig. 7). The results of FEM analysis confirm that adopted design and dimensions of element and rubber stiffness are satisfactory from the point of allowed stress and projected deflection.

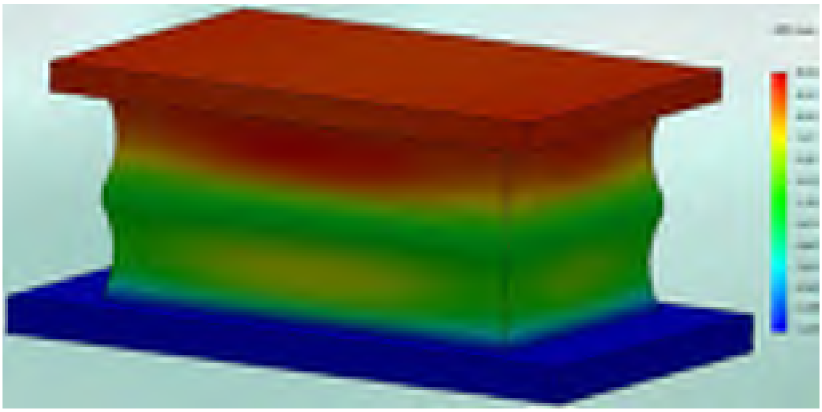

Figure 8 The deflection of rubber-metal elastic element

4 Exploitation tests

The exploitation tests of suspension with rubber-metal element was performed with the same measuring equipment and on the same track as previous test without it (Fig. 9). The change of measured deflection of rubber-metal element, as time function, for empty and laden wagon is shown in Fig. 10.

Figure 9 The exploitation tests of deflection of rubber-metal element

Based on the analysis of recorded signals of behavior of suspension, the quality of the introduced improvement was assessed. Characteristic loads of laminated spring with and without the rubber-metal element, in the static and dynamic conditions for laden wagon are given in Table 3.
From the Table 3 it is evident that the total static force on the one laminated spring F_u^{st} of fully laden wagon is lower for 48.4%. This means that part of the load is taken from the rubber-metal element, and in this way, even in the static conditions, the laminated spring is relieved for almost 50%. As expected, this was even more pronounced in wagon running at dynamic loadings.

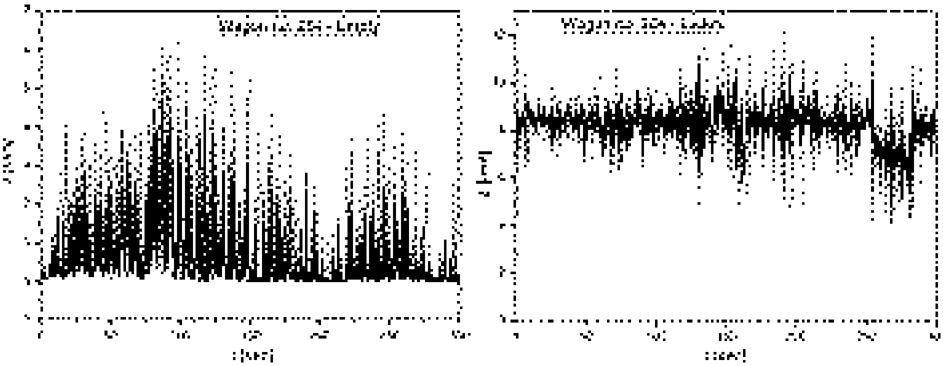

Figure 10 The measured deflection of rubber-metal element in exploitation

Table 3 The effect of the introduced improvement

Force on the laminated spring	Without rubber-metal element [kN]		With rubber-metal element [kN]	Relieving of laminated spring [%]
Static	F_u^{st}	119	61.42	48.4
Maximal dynamic	F_{max}^{d}	157	68.06	56.6
Minimal dynamic	F_{min}^{d}	125	53.45	57.3

The percentage of relieving of the laminated spring in dynamic conditions is increased and ranged between 56.6% and 57.3%. During the tests, the maximal dynamic deflection of rubber-metal element for laden wagon is equal to z_{max}=12.2 mm (Fig. 10).
Based on those obtained results, the rubber-metal elements were installed on over 400 wagons for coal transportation to the thermal power plant "Nikola Tesla" Obrenovac. During the later years of operation of these wagons there were no frequent fractures of laminated springs or other elements of suspension system. The number of fractures was reduced by more than 90%, where it should be noted that the rubber-metal elements are installed in the existing suspension systems. In mentioned period of 5 years there were no cases of debonding of rubber or any other defect of rubber-metal element. Of course, after this period, each element is replaced by new ones.

5 Conclusion

The fractures on the components of suspension are especially characteristic for freight wagons operating in extreme loading conditions. This causes a decreasing of the efficiency of rail transport and very often can cause a derailments with huge material damage and human victims. This this paper explore the possibilities for improvement of suspension behaviour through subsequent installation of rubber-metal elements of simple design. The methodology is based on the identification of causes of failures through numerical and experimental analysis of behaviour of suspension. Respecting the existing design of the wagon, the special solution of the rubber-metal element which can be subsequently installed in suspension can be designed. In this paper, the proposed methodology is applied on the freight wagons Fbd for coal transportation. The results show that static load of laminated spring of laden wagon is reduced by about 50%, while the dynamic load is reduced by over 60%. The number of fractures was reduced by more than 90%, and reliability of transportation is significantly increased. These results show that advantages of subsequent installation of rubber-metal elements in suspension of railway vehicles are very large. The proposed methodology can be applied even on the wagons whose suspension is not based on the laminated springs, but is based on coil springs or other elastic elements. The proposed solution implies that with the minimum processing and reconstruction, the existing suspension can be improved to the satisfactory level of reliability. The most important advantages of subsequent installation of rubber-metal elements in suspension is reducing of maintenance costs, increasing the efficiency of transportation, improving the ride comfort and running stability, etc.

References

[1] Huichuan Fan, Xiukun Wei, Limin Jia, Yong Qin: Fault detection of railway vehicle suspension systems, 5th International Conference on Computer Science and Education, pp. 1264-1269, 2010. ISSN: 1743-3533.

[2] Xiukun Wei, Hai Lui, Yong Qin: Fault isolation of rail vehicle suspension systems by using similarity measure, International Conference on Service Operations and Logistics and Informatics (SOLI), pp. 391-396, 2011. ISBN: 9781457705724

[3] Hayashi Yusuke, Tsunashima Hitoshi, Marumo Yoshitaka, Fault Detection of Railway Vehicle Suspension Systems Using Multiple-Model Approach, Journal of Mechanical Systems for Transportation and Logistics, Vol. 1, Iss. 1, pp. 88-99, 2008. ISSN: 1882-1782.

[4] Kumbhalkar, M.A., Yenarkar Y.L., Grover A.K.: Failure Analysis of Inner Suspension Spring of Railway Engine: A Case Study, Proc. of Int. Conf. on Advances in Robotic, Mechanical Engineering and Design, pp. 12-16, 2011. ISBN: 9781629934167.

[5] Stamenković, D., Radenković, S., Milić, M., Mladenović, S.: Gumeno-metalni elementi kod elektrolokomotiva, ŽELEZNICE, 2, str. 151-159, 1995. ISSN: 0350-5138.

[6] Stamenković, D., Milošević, M., Petrov, I., Banić, M.: Development and validation of elecrolocomotives primary suspension rubber metal elements, XIV Naučno-stručna konferencija o železnici ŽELKON 10, Niš, str. 79-82, 2010. ISBN 978-86-6055-007-3.

[7] Atmadzhova, D., Nenov, N.: Study on fatigue of rubber metal springs of primary spring suspension of electric locomotives, XV Scientific – Expert Conference on Railways Railcon 12, Nis, Serbia, pp. 41-44, 2012. ISBN 978-86-6055-028-8.

[8] Atmadzhova, D., Nenov, N.: Study of the Rubber Metal Springs Fatigue of Primary Spring Suspension of Electric Locomotives, Journal Facta Universitatis Series Mechanical Engineering, Vol. 10, No. 1, pp. 63-70, 2012. ISSN 0354 – 2025.

[9] Atmadzhova, D.: A method to evaluate the service life of rubber springs in rolling-stock, Academic journal Mechanics Transport Communications, iss. 2, pp. 2.9–2.14, 2008. ISSN 1312-3823.

[10] Petrovic, D., Bizic, M.: Improvement of suspension system of Fbd wagons for coal transportation, Engineering Failure Analysis, 25, pp. 89–96, 2012. ISSN: 1350-6307.

[11] Identification of the causes of the fractures of main leafs of laminated springs on the Fbd wagon, Study, Test Center of Wagon Factory Kraljevo, 1983. (in Serbian)

PLASTIC SLEEPER ANCHORS IN CZECH REPUBLIC

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Abstract

Sleeper anchors are used to increase lateral resistance of sleepers in continuous welded rail. Steel sleeper anchors are commonly used in the Czech Republic at present time. Chládek & Tintěra Pradubice company came up with the idea to use recycled plastic for the sleeper anchors production. The advantage of recycled plastic is less environmental impact and corrosion resistance. On the other hand problems with resistant to load were anticipated. This paper presents laboratory test of material of plastic anchors EVA V production. These tests were done during the developing of the plastic anchor shape. Similarly, in-situ tests were done on the trial test section to compare sleeper with plastic and steel anchors lateral resistance and sleepers without anchors lateral resistance are presented. For the supply of plastic sleeper anchors were parallel with the laboratory and in-situ tests compiled the technical specifications for control tests and where are listed all controllable conditions. These tests are described in this paper as well

Keywords: sleeper anchors, continuous welded rail, tearing resistance, lateral resistance

1 Introduction

Sleeper anchors are used to increase lateral resistance of sleepers in continuous welded rail. Steel sleeper anchors are commonly used in the Czech Republic at present time. Because plastic anchors are new product used in railway tracks and can affect the safety of operation to do many tests was necessary. Plastic anchors were primarily tested in the laboratory. In the laboratory was developed shape of sleeper anchors that was not problem with its production and ensure resistance to cyclic loading. After laboratory test plastic anchors were put into trial track section in the regional railway track with small radius curve. Plastic anchors in trial track section are monitored for a long time and also were made comparative tests of sleeper lateral resistance. For the supply of plastic sleeper anchors is necessary make technical specifications where all controllable conditions are listed.

2 Laboratory tests of plastic sleeper anchors and material properties

The Laboratory tests, aimed in evaluation of material properties and mechanical behaviour, were carried out in three stages. The EVA III design of plastic sleeper anchors was modified into the EVA V design after the first stage of laboratory tests. The design modifications include a new design of embedded steel parts without sharp corners and a new position of the casting point into the centre part of the sleeper anchor. Both modifications have improved anchor resistance against cycling loading as initial crack points were removed.
First of all the material from which plastic sleeper anchors are made was tested. The structure of the plastic sleeper anchor consists of the outer massive shell roughly 10 mm thick and the

inner relatively porous material as a consequence of the manufacture process. The samples for material tests were cut from the inner massive shell part of anchors. The objective of material tests was to measure basic mechanical properties – density, bending modulus and Charpy impact properties. Sensitivity to extreme temperatures – both low and high – was also inspected. Regarding the storage of sleeper anchors the flame resistance test was also carried out. The tests of material properties were performed in the Institute for testing and certification, a.s. Material properties were evaluated by following tests:

- material density by EN ISO 1183-1;
- bending behaviour by EN ISO 178;
- Charpy impact properties by EN ISO 179-1/1eA;
- Vicat softenenig temperature by EN ISO 306;
- determination of resistance to low temperatures – Charpy impact properties by EN ISO 179-1/1eA at temperature -10 °C;
- ignitability of products subjected to direct impingement of flame by EN ISO 11925-2.

Test results are summed in the Table 1. All tests of mechanical behaviour of plastic sleeper anchors were performed in Institute of applied mechanics Brno, Ltd., see [2, 3, 4]. The static load test was the basic test by which the mechanical behaviour was evaluated. This test simulates extreme load acting on sleeper anchors in track. An ultimate force that an anchor can resist is the result of this test. The tests were always performed to anchor destruction.

Table 1 Results of laboratory tests – material properties

Measured quantity	Value	Uncertainty
Density [g/cm^3]	1,02	0,01
Bending strength [MPa]	16,5	3,5
Bending modulus [MPa]	845	37
Vicat softenenig temperature [°C]	52,2	0,1
Charpy impact properties [kJ/m^2]	3,29	0,18
Charpy impact properties -10 °C [kJ/m^2]	2,32	0,09

The static load tests were carried out in the special loading frame. A sleeper anchor was attached to the steel element modelling the sleeper cross section. The force was applied by speed 2 kN/s, maximum speed of deflection was 5 mm/s. The force-deflection diagram was being recorded during the test and the anchor screw connection was visually inspected. Required force, which an anchor shall resist, is 12 kN with respect to the fact that minimum increase of the lateral sleeper resistance with anchor is 10 kN.

Tested samples were different during the stages of laboratory testing of plastic sleeper anchors. The first stage of testing (EVA III type of anchors) was performed only with new anchors that had just been manufactured. The last stage of testing was performed for new anchors, for anchors removed from test track section after 7 month of performance and for anchors which were put for 7 month into a climate box in temperature -18 °C. Termovision images were taken for deeply frozen anchors before and after tests. The results of static tests (average value) in last stage were:

- 21,4 kN for new anchors;
- 24,2 kN for anchors removed from the track;
- 22,4 kN for frozen anchors.

Figure 1 Assembly of static load test of sleeper anchor [2]

All results of the ultimate static force are significantly higher than required value 12 kN. Next important tests of plastic sleeper anchors were tests of resistance to cyclic loading, both regarding mechanical resistance and regarding abrasion. The parameters of the test of mechanical resistance were determined in aim to express the real loading of anchors in track. The particular test parameters were determined in accordance to administrator requirements:

- sin wave time course of applied force, min force in the cycle $F_{ZA,\ min} = 4$ kN, max force in the cycle $F_{ZA,\ max} = 10$ kN, the mean value $F_{ZA,\ mean} = 7$ kN;
- number of cycles $N_{ZA} = 2 \cdot 10^6$.

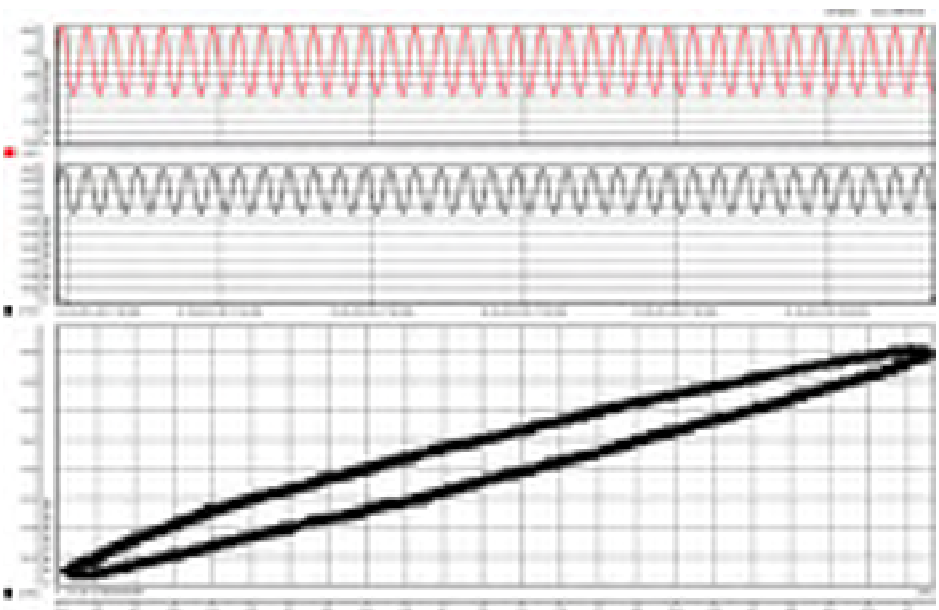

Figure 2 Test of cycling load resistence [3] (red color – force in kN, black color – deflection in mm)

Durability of plastic material (any cracks) and behaviour of the screw connection, especially tightness and squeezing the plastic anchor, were being inspected during the test.

The test to abrasion of plastic sleeper anchor was performed in the ballast box. The plastic anchor was loaded in vertical direction. The vertical displacement in loading cycles was determined in value 1 mm, which roughly corresponds with sleeper deflection under axle load. The ballast layer was checked every 12 hour to ensure full contact of the anchor and the ballast. The test parameters were:

· amplitude of vertical displacement $s_{ZA} = 1$ mm;
· number of cycles $N_{ZA} = 2 \cdot 10^6$.

Figure 3 Test of tearing resistance in ballast

The weight of anchor before and after the test was compared. The weight before test was 4,3120 kg, the weight after the test was 4,3118 kg so the weight loss was very small. A visual inspection of the anchor surface was carried out as well. The anchor surface was intact with only minor scratches caused by aggregates after the test. Plastic sleeper anchors resistance to abrasion in ballast was stated.

Figure 4 Plastic sleeper anchor after abrasion resistance test

Based on laboratory test results the use of plastic sleeper anchors EVA V in test track sections was approved. The test results were used for the declaration of material properties and mechanical parameters in the technical regulations of Railway Infrastructure Administration.

3 Technical specifications of EVA V plastic anchors

3.1 Material for EVA V sleeper anchor production

The EVA V type of sleeper anchor consists of a steel upper plate and an anchor segment. The anchor segment is made of material with a trademark of TRAPLAST which is manufactured by recycling of plastic waste. The surface layer of the anchor segment is comprised of not less than7mm thick compact material which creates a bearing shell. The sleeper anchors are used by mounting the upper plate and the anchor segment on sleeper then fixed by the use of two bolts and two self-locking nuts. Therefore there is a steel sheet in a flange of the anchor segment. It ensures a required contact with the fasteners. The material of the sleeper anchor is environment friendly, resistant to mechanical, climatic, chemical and biological effects and heat resistance.

3.2 Technical specifications

Supplier's technical specifications are obligatory technical specifications for delivery of the plastic sleeper anchors which are binding for the Czech railways. These specifications concern workshop production, testing, quality verification, acceptance, supply and use. The scope and frequency of the tests are prescribed by the technical specifications. Sleeper anchors EVA V properties and material quality for this product are proved by initial type testing repeated type testing and control product testing. The initial type testing is a test which is carried out before a new product is launched. It has been proved that the anchors meet the design requirements and parameters. The initial type tests of EVA V or its materials are:
· material properties for the anchor segment – bulk density, modulus of elasticity in bending, bending strength, heat resistance which is defined by softening temperature after Vicata, impact value after Charpy, fire-technical properties;
· lateral resistance of a sleeper with anchor (see chapter 4.1);
· ability of sleeper anchor to resist the loading required– static load test (see chapter 3.3);
· sleeper anchor ability to resist epeated loading – fatigue test, abrasion test (see chapter 3.5).

Repeated type testing is carried out if some changes, which could influence the properties of a sleeper anchor (for example in structure, material or shape), come to pass during large-scale production. The scope of the repeated type testing is determined by the user on the basis of changes proposed by the supplier. Control product tests are defined to verify continuously the quality of the product. These tests are ensured by the producer in scope and frequency according to the product management system or according to the inspection and test plans. Control product tests cover:
· dimensions of sleeper anchor;
· weight of sleeper anchor;
· bulk density;
· impact value after Charpy;
· static load test.

Expected lifetime of the sleeper anchor EVA V is 40 years. An intensified corrosion of steel parts might be expected in case of aggressive environment effect – the lifetime of the sleeper anchor might be decreased. The fasteners and the steel upper plate are possible to be coated with anti-corrosive paint.
Sleeper anchors EVA V don't require any maintenance. Regular control of fasteners is recommended especially in case of aggressive environment.

3.3 Static load test

Maximum force on limit loading capacity of the anchors is monitored during a static load test. Deformations of a trial sample depending on an acting force are monitored. The static load test is carried out in a loading frame on a model of sleeper anchor. A sleeper is replaced with a steel model. The shape of the model corresponds with the lateral cross section of the sleeper in the place of anchor. Bearing area of the sleeper's model has a shape of a rectangle. The length of the bearing area corresponds with the width of an underneath surface of the sleeper. The width of the bearing area is 200 mm. Roughness of the bearing area which is in contact with the plastic anchor segment corresponds with roughness of the underneath surface of the sleeper. The biggest unevenness mustn't be higher than 1 mm. The upper steel part surrounds the sleeper's model and the anchor segment fits to the bearing area. For arrangement of the test see Figure 5. The test is carried out on three trial samples which are loaded until destruction. A loading rate is 2 kN/s.

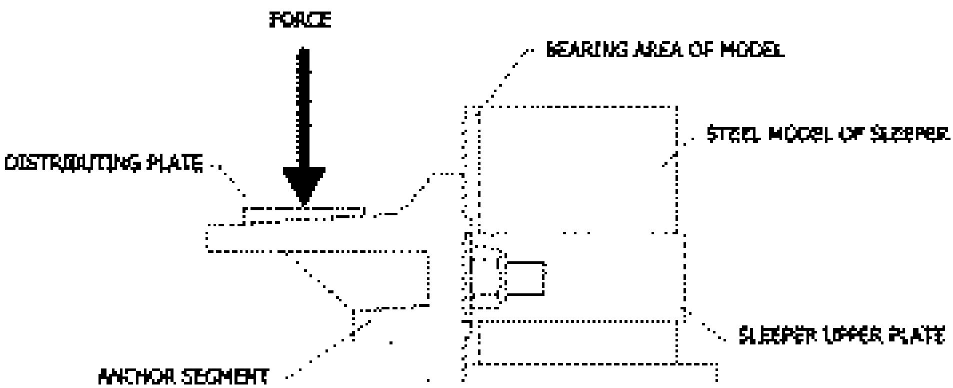

Figure 5 Static load test arrangement

3.4 Long-term fatigue test

Fatigue test examines the ability of the anchors to resist repeated loading. The fatigue test is carried out in a loading frame on the same model as the static load test (see Figure 5). One trial sample is loaded by a cyclic loading from 4 kN to 10 kN. The loading is introduced with a frequency range from 2 to 5 Hz. Number of load cycles is $2 \cdot 10^6$. Holding power of screw joints, lifetime of the plastic part and a squeezing of plastic are controlled visually during the fatigue test.

3.5 Abrasion test

An abrasion test monitors the ability of plastic material of the anchor segment to be resistant to cyclic loading. The abrasion test is carried out on a model of a sleeper with a sleeper anchor in a ballast box. Crushed rock of fraction 31,5/63 is placed on a solid base – concrete or steel plate. Sleeper is substituted for a steel model. The bearing area of the steel model represents an underneath surface of the sleeper. It has a shape of a rectangle. Its longer side corresponds to a width of the underneath surface of the sleeper. Its shorter side has a length of 200 mm. Roughness of the bearing area which is in contact with the plastic anchor segment corresponds to roughness of the underneath surface of the sleeper. The biggest unevenness has to be less than 1 mm. For the test arrangements see Figure 3.

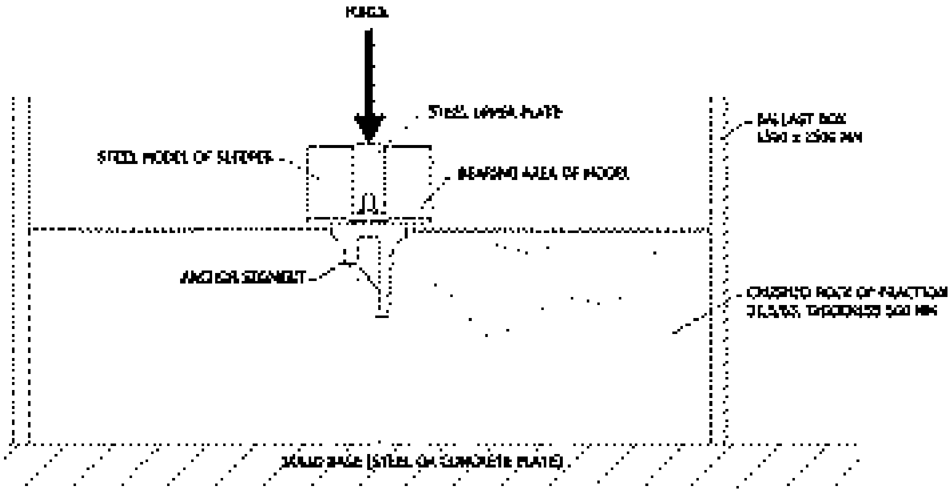

Figure 6 Abrasion test arrangement

The test is carried out on one trial sample. The Intensity of force is chosen according to the vertical movement of the sleeper anchor – maximum amplitude ±1 mm. The introduced loading has sinus course with frequency range from 2 to 5 Hz. Number of load cycles is $2{\cdot}10^6$. Weight loss of the anchor segment is controlled and is determined from the weight before and after the abrasion test. Surface of the sleeper anchor is controlled visually after the test.

4 Tests in trial test section

The trial test section was set up in regional track Svitavy – Polička in the Czech Republic. It is circle with radius 200 meters, cant 75 milimeters and 32 meters long transition curves. Superstructure consists of continuous welded rails S 49 on concrete sleepers with ribbed baseplates. The ballast bed is gravel fractions of 31.5 / 63. Ballast bed was changed only in the upper layer, from the underneath surface to the top sleeper, during the track reconstruction. Ballast bed under sleepers is very dirty. The shape of the ballast bed is prescribed with enlargement (1.75 m) and overcutting (0.1 m) on the outer side of the curve. There is sleeper anchor on every sleeper in circle. There are plastic sleeper anchors in first half of circle and steel sleeper anchors in the second half of circle.

4.1 Lateral resistance test

Measurement preparation starts by removing ballast behind sleepers head (rear in the direction of extrusion) in order to put the extrusion head. Next, remove the sleeper screw in the baseplates of both rails. Both rails are lifted, but just to be pulled away the pads from sleepers. For the lifting of rails is necessary to loosen the sleeper screw and the neighboring sleepers as well. Extruder head is put on the end of sleeper and the device is aligned by supporting rod. Prepared extruder device is shown in Fig. 7.
Displacements are measured against 1.5 meters long fixed frame above tested sleeper. The force is set into the system by hand operated hydraulic cylinders with pressure gauge. Measurement is performed by shifting sleeper in 1 millimeter increments, and each shift values are assigned values read from the pressure gauge. Dependence shifting on the load power is gotten. After steady resistance sleeper, which are reflected by shifting sleeper without the increase the strength force decrease to zero.

Figure 7 Extruder device

During the test the lifting sleepers is undesirable. The test arrangement with disassembled sleeper screw and by minimum lifting the rails with baseplates, lifting sleeper is impossible. After the test is necessary to sleeper move back to the starting position – after unloading remain displaced. It is not possible to repeat the test on the same fret.

4.2 Test in the trial test section

In-situ test were done in two stages – during reconstruction one day after continuous welded rail installation and after 8 month operation. In both stages were tested 10 sleepers – 5 with plastic sleeper anchors EVA V, 3 with steel sleeper anchors and 2 without any sleeper anchors for comparison. Sleepers without anchors are in transition curve because all sleepers in circle are with sleeper anchors.
Between the tested sleepers were always 3 sleepers. Rails were always released only at the measuring sleeper and the immediately adjacent sleepers, which was particularly advantageous due to temperatures. Immediately after the test rails were clamped and then the work began to prepare for further examination. Rails have never been released on more than 3 sleepers and could thus lead to changes in the clamping rail temperature.

4.3 Test routine

After installation extrusion system to tested sleeper was initiated by raising the pressure and ejecting sleepers. Shift sleeper was measured by two displacement sensors. The measured values of both sensors were averaged. The pressure was increased to a steady value. In the case where stabilization occurred the test was finished after 15 mm displacement.
Figure 8 was created by averaging the measured values for sleepers with plastic, steel and without any anchors in both testing stages. There are sleeper resistances similar in both stages of the measurement for both sleepers with EVA V plastic anchors and sleepers with steel anchors. To change the inclination of the curve occurs at a displacement of about 1 mm and a transverse resistance of 7-8 kN, respectively 9-10 kN. In the case of steel anchors there is a change of inclination at a slightly lower resistance value, but the increase in lateral resistance is more significant.

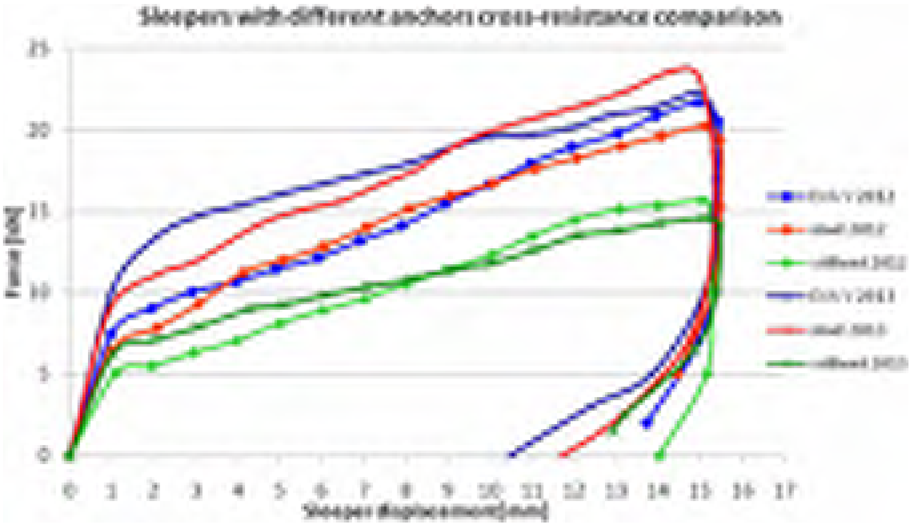

Figure 8 Lateral resistance comparison

5 Conclusions

Plastic sleeper anchors have been developing in several steps with both laboratory and in-situ tests. The latest version plastic sleeper anchors EVA V is in trial operation in regional railway track Svitavy – Polička in the Czech Republic. On the basis of laboratory test and positive experience with trial test section seems using plastic sleeper anchors EVA V as good alternative steel sleeper anchors. Another advantage of the plastic anchor is the use of more environmentally friendly recycled plastic.

Acknowledgment

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References

[1] Institute for Testing and Certification, a.s.: Test report No. 412205120, Zlín, 2011. 5 pp

[2] Institute of Applied Mechanics, Ltd.: ZP4886/11 Tests of plastic sleeper anchors EVA III. Brno, 2011. 6 pp

[3] Institute of Applied Mechanics, Ltd.: Test report No. 5024/12. Strength tests of EVA V plastic sleeper anchors: Static load test and longterm fatigue test. Brno, 2012. 13 pp

[4] Institute of Applied Mechanics, Ltd.: Test report No. 5374/13. Mechanical tests of EVA V sleeper anchors. Brno, 2013. 44 pp

[5] Brno University of Technology, Faculty of Civil Engineering: Lateral resistance tests of sleeper with plastic sleeper anchors, Brno, 2013. 10 pp

ROLLING CONTACT FATIGUE ON TRAMWAY'S RAIL

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Abstract

Head checking cracks as a special type of rail defects become more frequent recently on the high-speed railways. Partly similar defects were observed at the starting and stopping locations of the vehicles' driving axle at urban tram stops. In these places fatigue defects are appearing parallel to the track on the rail's running surface. These defects were first observed by the author of the abstract in Budapest, along tram Line 49. The most significant defects were discovered on sections of that tram line, where only old carriages run without slip protection equipment. On those sections, where other types of the carriages also run, the defects were less frequent. Measurements of eventual defects were performed using Eddy current sensor, digital microscope, wheel mounted inertial sensor and high-speed camera. Measurements with Eddy current sensors were carried out on the running surface of rail. They did not show cracks but average depth of defects could be determined. Surface deformations were detected by a digital microscope and their dimensions were also measured. Images made by digital microscope (from "micro-slip") showed similarity to the "comet-shaped" rail surface corrugation caused by high-intensity acceleration ("macro-slip"). Based on this fact, the author assumed that probably the slip plays an important role in the formation of defects. Slip values recorded by a high-speed camera during start and stop of tram carriages were found equivalent to the length of the defect under consideration. Further investigations were made to determine the exact value of the slip during the start and stop of the tram carriage under operating conditions. For that purpose, wheel- and vehicle-mounted accelerometers and the speed acquisition device of the tram carriage were used. Assessing the results of the experiments, it is concluded, that the wheel slip is responsible for the defects discovered, but further investigations are needed.

Keywords: rolling contact fatigue defect, wheel slip, wheel-rail contact, special rail corrugation, micro-slip

1 Introduction

Some special defects were observed during a condition survey of Budapest tram lines carried out by the author. This condition survey was based on consultations with professionals of BKV (Budapest Public Transport Plc.) and photos taken during the perambulation of tram lines and the performed investigations. Fig. 1 shows some typical defects.
One of the observed defects was distinguished, in order to analyse its causes in detail. This defect is a special corrugated longitudinal one, attributable to rolling contact fatigue appearing on the rail tread. Fig. 2 shows it in Sections 1.1.

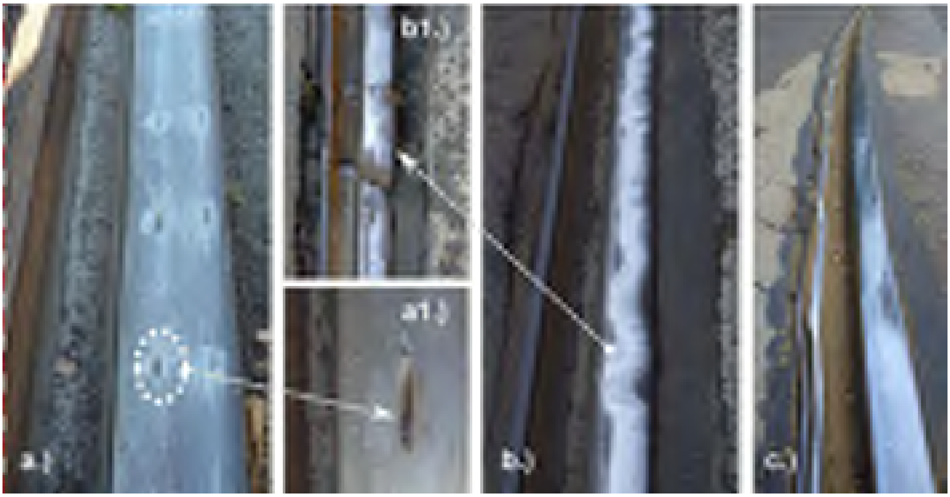

Figure 1 Special rail corrugation and defects: a) The pitch of rail corrugations was cut by electromagnetic rail-brake; b) "Winding-shaped" wear on the running surface, later broken; c) Special corrugated rail side wear of grooved head in curve

1.1 Longitudinal defects appearing on the rail's running surface

Defects apparently similar to head checking cracks (HC) were observed at starting and stopping location of the driving axle at tram stops. At these locations fatigue type defects appearing on the running surface are parallel to the track. It should be noted that longitudinal surface defects occur most frequently on the full width of rail head (Fig. 2c). However, the defect in other rail sections has a shape of corrugation (Fig. 2a). It shows similarity to the impact of unstable high-frequency bogie movement which can occur mainly on rails of high speed railways. The wheel probably slips at the moment of start and stop. This slip leads to the hardening of the rails' running surface which results failures occurring on the rail tread due to cyclical fatigue load. This type of defects was first observed by the author in Budapest, along tram line 49. The most significant defects were discovered on sections of that tram line where only aged carriages run, without slip protection equipment. These defects appeared less frequently on sections where other types of tramcar also run.

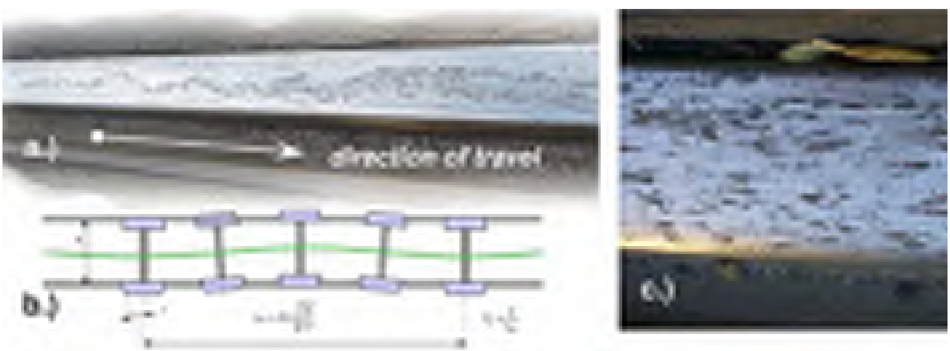

Figure 2 a) Corrugated shape of failure; c) Longitudinal fatigue defects on the whole cross-section of running surface

It has to be noted, that both, TATRA and GANZ made rigid-axle articulated tramcars run on the studied tram line 49, but the latter type does not have slip protection equipment and torque control in the engine. Aiming to discover the characteristics of the defects, the following measurements were performed.

1.1.1 Testing of rail surface deformation using Digital microscope

Images of digital microscope showed inelastic deformation ("micro-slip"). The microscope was controlled by a computer, which immediately recorded the photos taken with appropriate lateral overlap, so the microscopic pictures can be joined easily by using an appropriate software. Fig. 3c shows a panorama photo of the discovered defect. Surface deformation of the displayed microscopic failure (Fig. 3c) shows similarity to the "comet-shaped" rail surface macro corrugation caused by high-intensity acceleration, where the direction of the movement can be also determined easily (Fig. 3b).

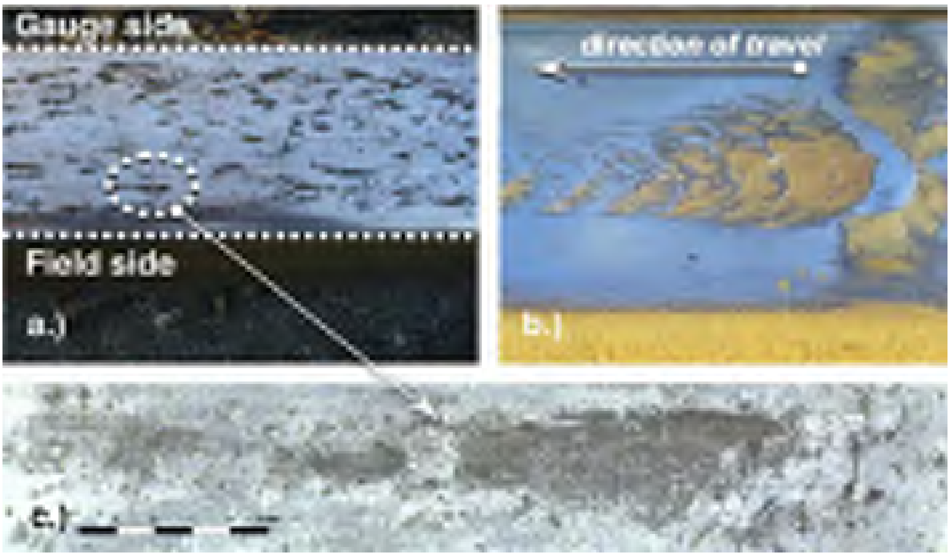

Figure 3 a) The longitudinal failures; b) macro "comet"- shaped defect on the rail running surface c) Panorama photo of digital microscope

Geometric dimensions of a defect could be also determined from the pictures taken by a digital microscope. The length of the observed failure is varying between 5 and 25 mm (Fig. 3c) and the dominant wavelength of the displayed wave-like defect shown in Fig. 2a is between 0.10 and 0.30 m. A difference exceeding two orders of magnitude appears when this value is compared to the theoretical wavelength derived from the calculation based on Klingel's formula [3] Eq. (1):

$$\lambda = \frac{2\pi}{\sqrt{\frac{2tg\gamma_e}{e \cdot r_0}}} = 14.1\,\mathrm{m} >> 0.1 - 0.3\,\mathrm{m} \tag{1}$$

where:

$tg\gamma_e$=0,05=1/20	Equivalent Conicity = wheel profile cant (1:20);
e=1500 mm	Nominal rolling radius distance;
r_o=670 mm	Nominal rolling diameter.

This result can't be explained by the traditional sinusoidal motion theory. It should be noted that Klingel's theoretical wavelength refers to the motion of one single wheelset rather than the motion of the whole carriage. For the sake of the calculation, it was assumed, that the wheel and rail profile are to be in perfect condition and shape. Although smaller values of wavelength can also occur if the profiles of the wheel and rail are worn out. Values of wavelength obtained from Klingel's formula are actually falling between 1 and 40 meters.

1.1.2 Detection of cracks in railhead by using Eddy current sensor

Eddy current sensor is used to detect and determine the depth of crack-like damage in the gauge corner of the rail. Devices applied for this measurement are continually improving, so cracks under the running surface can be detected in a reliable way up to a depth of 3-4 mm by using this technique, depending on the Eddy current excitation frequency. Determination of crack's depth is very significant for traffic safety and for specifying the maintenance work to be carried out on the rails of high speed railways. Identical method was used by the author to discover cracks under the running surface near the observed defects.

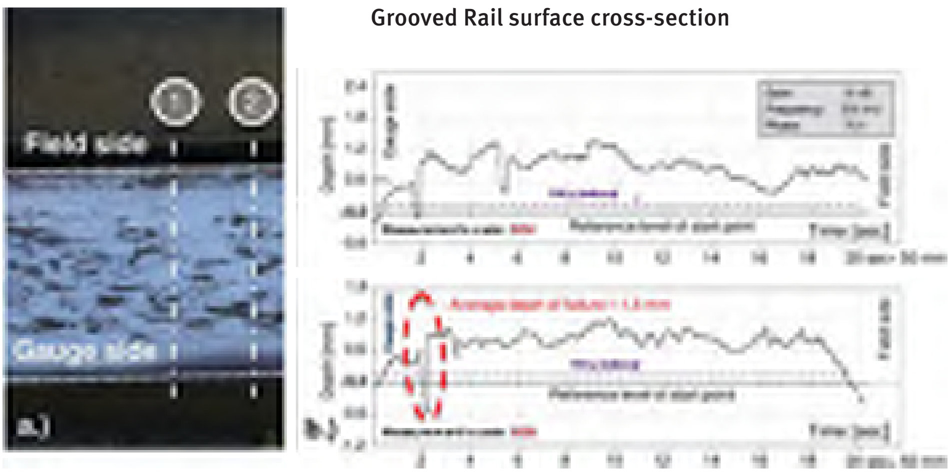

Figure 4 Testing result of two cross-sections by using Eddy current sensor

Measurements were performed using Hocking Locator 2, a single frequency device. The Eddy current test method is based on the principle of magnetic induction. The primary magnetic field of the excitation coil (sensor) induces a secondary magnetic field in the rail. Measuring the changes of this magnetic field the location of sub-surface cracks and material failures can be determined. The sensor is moved with constant speed transversally on the rail's running surface (the width of a rail head is measured in 20 seconds). This timebase mode crack detection test was performed in those cross-sections, where the size of the defects was the greatest (Fig. 4a). In the time-base mode the Y component (depth) is represented against time. Near-surface cracks were not discovered by using Eddy current sensor.

1.1.3 Investigation of tramcar wheels

The intensity of the use of sand to prevent slip by increasing friction can play a significant role in the appearance of the defects studied. In order to check the wheels of vehicles running on the tram line 49, the tram depot was visited in order to find out whether the sand poured under the wheels, could be the cause of these defects. However, it was stated, that no crack-causing matter has been poured on the running surface of the wheel other than some contaminations in the sand itself. No wears were observed on the wheel, unlike the rail heads, as described above.

2 Wheel-slip testing

The results of these investigations (Sections 1.1.1 – 1.1.3) lead to the conclusion that the wheel micro slip (Section 1.1.1) can play a significant role in the appearance of the defect described above (Fig. 2), therefore the slip values were determined at the moment of start and stop of the tramcar, under operating conditions. The tests were performed on a back storage track of tram depot in Budapest under dry, cold weather and good adhesion condition. The tramcar used for the test runs on that line, where the observed failure is situated. (Section 1.1 refers to main parameters of the tested tramcar). Scale signs were marked up on the wheel and along the track before the measurements, therefore the slips become easily determinable by visual observation (Fig. 5).

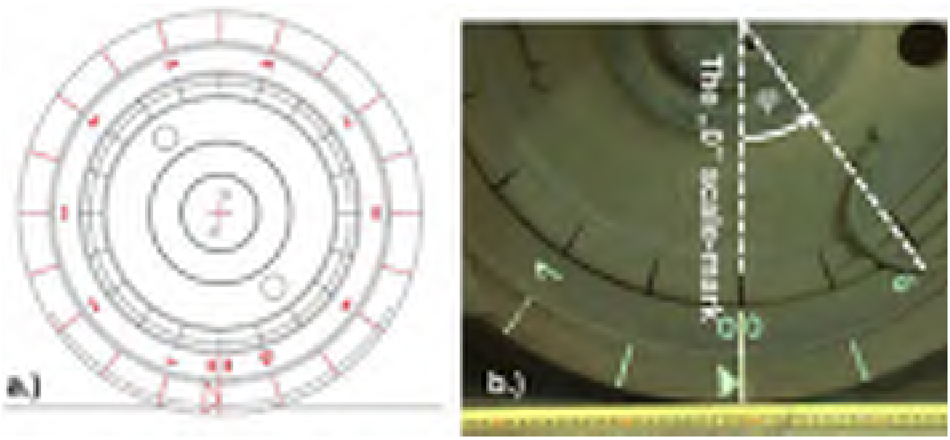

Figure 5 a) Regular scales marked on the wheel; b) the "0" scale-mark and the scale-marks placed alongside the track

2.1 The process of testing at the moment of start and stop

Testing at the moment of start: The tramcar was started according to operating condition with different intensities of acceleration, and then it was run out without braking to end of the track until it stopped. It was slowly driven back to the starting position, using only electromagnetic rail brake to stop.
Testing at the moment of braked stop: The vehicle was started from the other (side) end of the track and it was coming towards the HS camera with maximum speed (30 km/h) according to the local conditions, and it stopped using the maximum intensity of the brake. The vehicle was stopped in the visual field of the HS camera.

2.1.1 Test using visual observation and high speed camera (HS)

The position of the "0" scale-mark on the wheel related to the scale-marks placed alongside the track was read at the start location and also when the vehicle returned that same position (Fig. 5b). The slip value is the difference between the two readings. The test by using HS camera is in principle similar to that of the visual observation, but in that case the position of the "0" scale-mark is read by the HS camera. Time – distance data points of the wheel axle, and the time – arc-length data points of the angular displacement of the "0" scale-mark are determined by graphical analysis of the video recorded by the HS camera. Quadratic Least Square Regression was performed to data points where the correlation coefficient related to the set of all cases was higher than 80%.

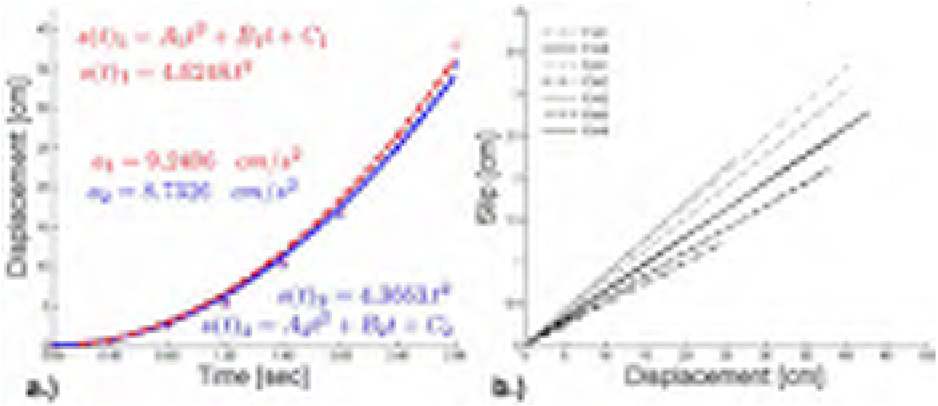

Figure 6 a) Time – distance, and time–angular displacement functions in case of starting by high intensity acceleration; b) distance – slip diagram in case starting by various intensity acceleration

Fig. 6a shows the parameters of the fitting curves in case of a start with high intensity acceleration (the Quadratic Regression Equation = ax^2). The residual values (difference between the fitting curve and the actually observed values) are caused by imperfection of reading from scale-marks alongside the track. The slip value (Δs) is determined by the difference of fitting curve Eq. (2):

$$\Delta s = s(t)_1 - s(t)_2 = A_1 \cdot t^2 - A_2 \cdot t^2 \qquad (2)$$

The measured and calculated slip values at the start reflected micro-slip that is directly proportional to distance (Fig. 6b). This could be caused by the linear engine of the tramcar, which assures uniformly accelerated motion.

2.1.2 Test using wheel – mounted accelerometer

Wheel slip testing can be improved by using wheel-mounted digital 3-axis accelerometer [2], ensuring continuous monitoring of the wheel motion (slip). This can significantly reduce the uncertainty of the previous methods (refer to Sections 2.1.1.). The accelerometers are mounted on the driving wheels and the free running wheels too. The longitudinal axis of the device is perpendicular to the wheel radius. The magnitude and the location of the slip can be determined from the measured acceleration value (Fig. 7). The process of starting and stopping corresponds to the descriptions in Sections 2.1. This methodology is worked out by the author in MATLAB software by analysing acceleration value of the accelerometers that are mounted on the wheels of bicycle.

The accelerometers are placed preferably in identical position on the wheels (they should be in equal phase – cophasal -) before starting. The magnitude of wheel slip can be determined from the “pre-start” and “post-stop” angle differences of the accelerometer positions by using accelerometer values (a_x, a_y) (Fig. 7). The directly measured acceleration data is distorted by noise, while that latter could be separated from the data by using linear-phase low-pass digital filter [4] in MATLAB. Fig. 8 illustrates the filtered signal and the acceleration, braking and coasting sections in case of a starting with low-intensity acceleration. The magnitude and location of the slip is determined from the phase shift of the measured accelerometer data read on different wheels. It can be noted that the calculation must be corrected, because of the difference between the accelerometer run circuit and the length of tread. Further tests with a properly equipped tramcar are planned to be performed in spring 2014.

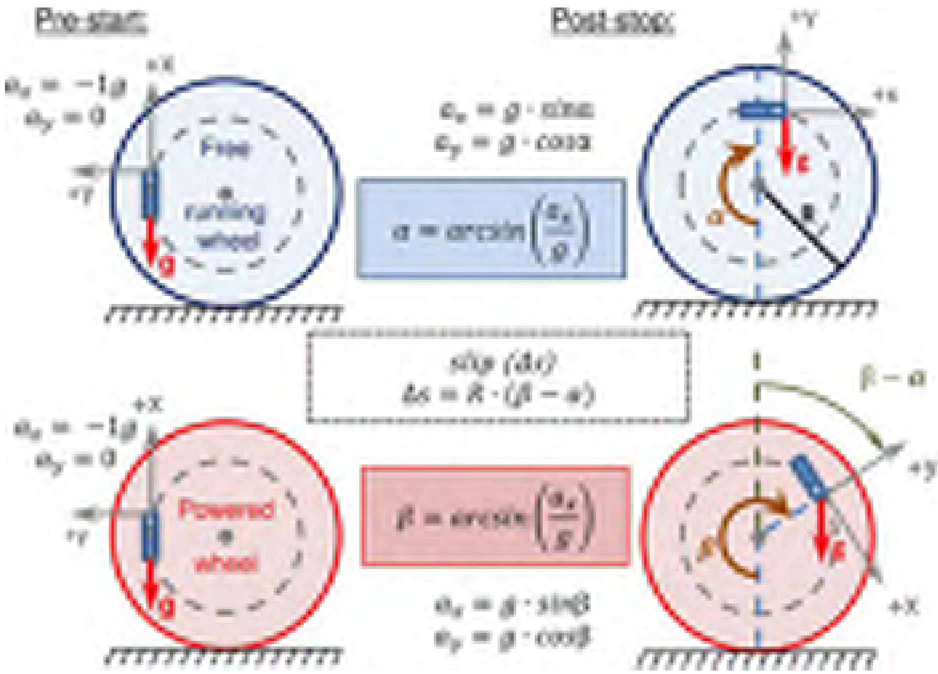

Figure 7 Wheel slip testing by using 3-axis accelerometer

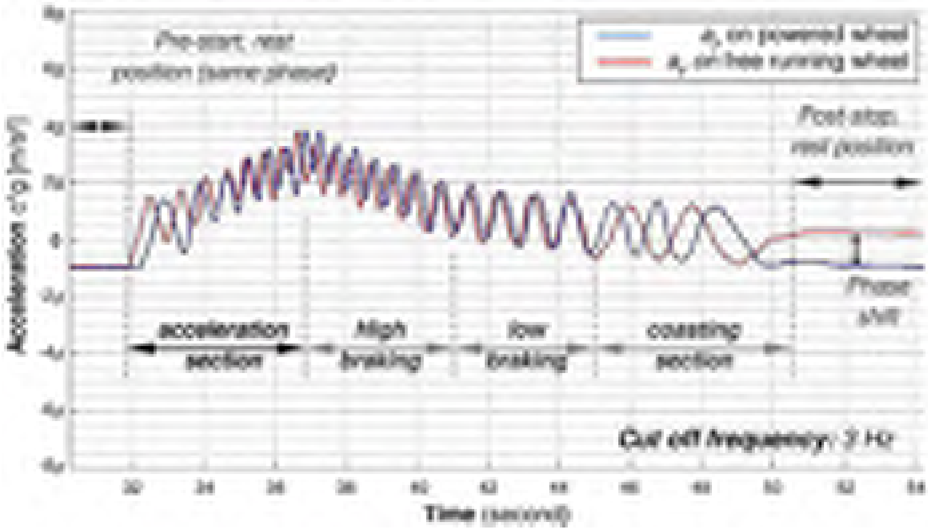

Figure 8 Centrifugal acceleration of powered and free running wheels

3 Summary

The most significant defects were discovered on those sections of the tram line stops, where the driving bogies stop. Surface deformations discovered (Fig. 3c) showed similarity to the "comet-shaped" rail surface macro corrugation, caused by high-intensity acceleration. During wheel slip testing, the tramcar does not slip in a visually perceptible manner, but the detailed investigations showed micro-slip directly proportional to distance. Furthermore, the lengths of the defects were equal to the measured slip values at the moment of start. On the base of these facts it can be assumed, that probably the slip plays an important role in the formation of these defects. The uncertainty of the wheel slip testing is reduced by applying wheel-mounted digital 3-axis accelerometer, which ensures the continuous monitoring of the wheel motion. Although the measurement methodology has already been developed, actual measurements on the field are planned to be performed in spring 2014.

References

[1] Esveld, C.: Modern Railway Track, Second edition, TU-Delft, 2011.

[2] Gersdorf, B., Frese, U.: A Kalman Filter for Odometry using a Wheel Mounted Inertial Sensor, 10th International Conference on Informatics in Control, Automation and Robotics, Reykjavík, pp. 388 – 395, 2013.

[3] Klingel, J.: Über den Lauf von Eisenbahnwagen auf gerader Bahn. Organ für die Fortschritte des Eisenbahnwesens, pp. 113–123, 1883.

[4] Simonyi, E.: Digitális szűrők (Digital Filters) Műszaki Könyvkiadó, Budapest (Hungarian Technical Press, Budapest), 1984.

6 STRUCTURAL MONITORING AND MAINTENANCE

BRIDGE EVALUATION METHOD USING METROLOGICAL METHODS IN SHORT AND LONG-TERM MEASUREMENTS

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Abstract

Bridges are an essential component of the train and vehicle traffic network. Increasing traffic loads, the demand for faster transit speeds and the possible influences of weather effects drive the need to evaluate existing bridges that in some cases have been in use for decades. The existing construction documentation is not always sufficient to obtain reliable statements about e.g. the remaining useful life or changing utilization conditions. Suitable metrological methods combined with calculation models allow these tasks to be resolved.

Keywords: bridges evaluation, metrological methods, short–term measurements, long–term monitoring

1 Introduction

The Deutsche Bahn (DB) alone is responsible for approx. 28,000 railway bridges with diverse structural designs. Bridges age and fatigue differently depending on their construction. Even identically designed bridges evidence different behaviors as e.g. climatic conditions on site may vary. Deutsche Bahn generates calculation reviews at various levels, in compliance with Guideline 805 [1]. This frequently means a measured-value supported calculation review in the case of complex bridge structures [2]. The measured values, with which the calculation models are calibrated, are provided by short-term measurements with resulting loads from standard traffic or defined special loads. The real system behavior can be acquired easily by measurements on the object. Short-term measurements highlight weak points and safety deficits. Long-term measurements depict the behavior of the bridge under load collectives which include, e.g. in addition to traffic loads, wind forces, temperature fluctuations and other influences.

2 Short–term measurements for measuring system behavior on bridges

Static or dynamic short-term measurements as per Module 805.0104 (DB) are classified as follows:
- System measurements under defined operating loads for steel railway bridges;
- Experimental load-bearing evaluation (concrete constructions with high non-linear load-bearing behavior);
- Special measurements to clarify the behavior of e.g. moving bridges.

A bridge under investigation is experimentally evaluated with both general train traffic loads and traffic with some extent significantly higher loads. The necessity of increased loading is often justified by the aim of obtaining a measurable construction reaction (strains, curvature, shifts, inclinations, accelerations, etc.). The loads however are only located in the elastic deformation behavior of the construction and are specified by the bridge evaluator/structural engineer. He also defines the number and type of measurement points in the respective measurement cross-sections and the type of dynamic load by e.g. defined crossings of vehicles at different speeds. In this method, the number of measurement points can lie between double figure and several hundred applications. The duration of the measurement is frequently limited to just hours (closing sections; provision of load testing vehicles; special wagons). This defines the requirements for the measurement technology through the dynamic and simultaneous acquisition of measured values.

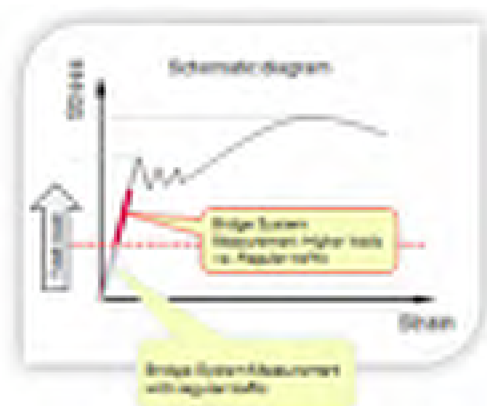

Figure 1 Schematic representation of the applied loads during a system measurement

3 Measuring the system behavior on the Fehmarnsund Bridge (Example Project)

3.1 Application and requirements

Arising from the discussion and the initial constructional proposals for the building of the Fehmarn Belt link (Fig. 2 – red arrow) from the Fehmarn Island (Germany) to Denmark, the Fehmarnsund Bridge (Fig. 3), which was built in 1963, was also subject to a new evaluation. This network arch bridge, located to the south of the planned Belt link (yellow arrow), will experience significantly greater use with a direct traffic connection, both in terms of traffic density and with regards to the forecast traffic load.

Figure 2 Geographical location

Figure 3 Network arch bridge (since 1962), Fehmarn Belt Bridge

The system behavior measurements between 11 and 14 June 2010 were intended to determine whether the structure could meet these requirements. The assessment of the resulting measurement results is not a part of this document. The basis of the assessment is the evaluation level 4 of the Guideline 805 and the evaluation guideline of the Federal Highway Research Institute. Numerous measurements with various loads were necessary on the road and on the rail tracks for calibrating the complex calculation models. The measurement program was agreed with the structural engineer group of the DB-Projektbau [3].

3.2 Measurement program

Figure 4 Measurement locations and corresponding measurement sections

It was agreed with the structural engineers that two measurement locations (ML I and ML II), with corresponding measurement sections, should be defined (Fig. 4). ML I comprises two field sections and a support section. ML II is located on the network arch bridge superstructure and has two measurement sections.

3.3 Type and installation of the transducers

Sensors were installed at 251 locations representative of the bridge statics to acquire diverse bridge construction deformation parameters under cyclical stresses. The specified measuring points were installed in the period from April to June 2010. The specific number of measuring points was based on the use of strain gauges (SGs) in an SG full-bridge circuit. The strain to be measured as reaction to the applied loads is acquired in a full-bridge branch (component expansion + temperature expansion, Fig. 5).
This quarter branch is connected with three passive SG to form the full bridge circuit. These passive SG do not experience any component expansion caused by applied loads like the SG in the quarter branch, but they acquire the resulting temperature expansions.
This circuit type significantly minimizes the acquisition of temperature-dependent strains so that only component strains in correlation to traversing events are acquired. The SG full bridge circuit type enables electrical connection to the downstream measurement electronics in a 6-wire circuit. Cable effects which could result in metrological errors, caused e.g. by temperature influences on long cables, are compensated for by the electronics here via two additional sense leads.

Figure 5 Strain gauge installation

3.4 Measurement data acquisition

Data acquisition systems were installed decentralized for measurement data acquisition, storage and transfer. Due to the dynamic acquisition of the measured values, the decentralized installed data acquisition systems (HBM QuantumX and MGCplus) and the main control room (Fig. 6) were connected with fibre-optic cables for NTP data synchronization and controlling the measurement system. The dimensions of the measurement layout itself and the complexity of the measurement task can also be indicated by noting that the use of fibre-optic cables replaced 60 km of electrical connection cables that would have otherwise been necessitated.

Figure 6 System architecture

3.5 Measurement implementation

Appropriate test loadings on the Fehmarnsund Bridge were implemented with locomotive units and additional heavy truck transports between 18:00 to 06:00 each night between 11.06. to 14.06.2010.

3.5.1 Quasi-static transits of heavy-load vehicles

Two heavy-load vehicles (Fig. 7) were used for transits in 4 different lanes (axis load 12t / 20t) at speeds v = 10 km/h and a vehicle distance of approx. 10 meters.

3.5.2 Locomotive train transits

The locomotive unit (Fig. 8) comprised a group of 2 x BR232 and 8 x BR155. The total mass was 123t with an average wheel set load of 22.5t for a BR 155 locomotive. The basic load position resulted in a vehicle mass of 6.276 t/m.

Figure 7 Heavy transporter with 120t additional load

Figure 8 Load train comprising 10 locomotives

3.5.3 Combined transits

In this part of the measurement program, combined transits of locomotive units and heavy transporters on the routes were implemented with different travel directions and load patterns.

3.5.4 Dynamic measurements

Dynamic transits were implemented at maximum speeds for the locomotive units of approx. 120 km/h and approx. 80 km/h for the heavy transporters. The measurements were triggered by light barriers positioned at suitable points.

3.5.5 Measurement results (example)

Fig. 9 shows an example of the mechanical stresses acquired during parallel transits of locomotive units and heavy transporters during one of the implemented measurements.

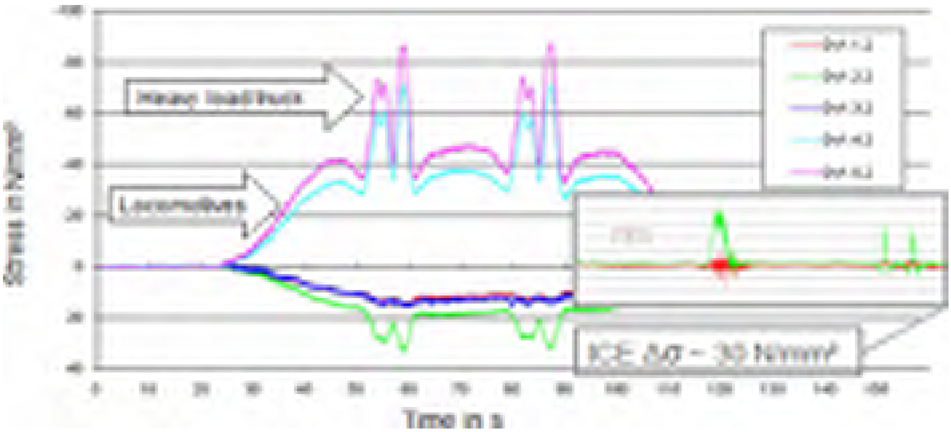

Figure 9 Stress ratios on five field continuous girder – Field center of first field transverse beam Section 2 (guide barrier) with parallel transit of locomotive unit and heavy transporter at measurement point 1 to scale copied comparison (ICE standard traffic)

4 Long–term monitoring on bridges

Long-term measurements provide statements about the resulting stress collectives of a structure under the influence of standard traffic, in combination with all other physical influencing factors on the structure that do not e.g. occur in short-term measurements. Structural health monitoring detects changes in time and helps to avoid accidents. The measurement data shows how structural components interact under real loads. The know-how thus obtained delivers important information for the maintenance of existing bridges and for the development of innovative construction methods. Monitoring is useful when the investment volume only permits a limited number of new structures per time unit, where historical structures need to be maintained and continue to be used, and where special constructions require permanent monitoring.

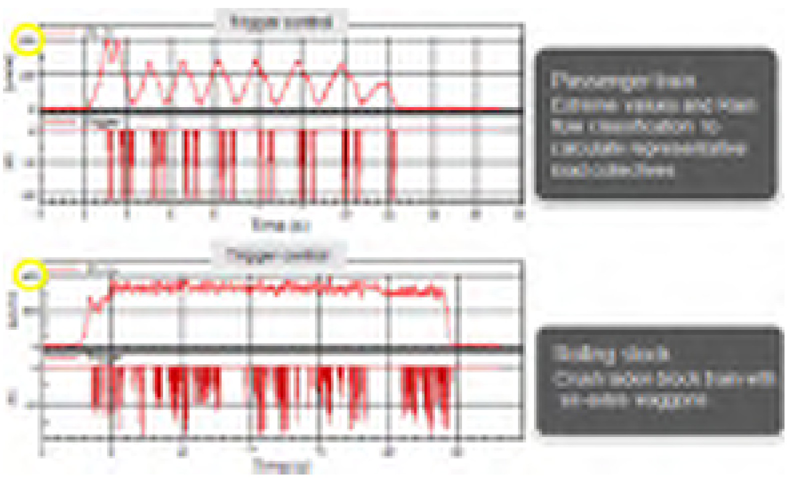

Figure 10 Permanent monitoring of resulting mechanical stresses on a DB auxiliary bridge, comparing stress of passenger and freigth trains

Figure 11 Typical design of a long-term monitoring system (data transfer) of the DB in cooperation with HBM

5 Long–term monitoring for remaining life statements – Status and prospects

The basic requirement for measurement technology-supported statements about the remaining service life of a construction (bridge) is the availability of reliable load data from the past. In part, these can be calculated on the basis of available documents about train loads and the number of trains or projected using information available about loading situations with reference to defined time periods. If such data is not available, it is not possible to make statements about the remaining useful life! Challenges are not only posed by changing climatic conditions, but also with regards to protection against vandalism and theft, if future solutions are to have a direct influence on standard traffic and the utilization analysis of railway bridges.

Aim of existing pilot projects:
- Long-term stability and durability of sensors and subsequent electronics;
- (e.g. endurance strength/protection of sensors against weather, overvoltage protection, etc.);
- Maintenance free overall system/secure remote data transfer;
- Remote configuration and analysis option;
- Redundancy at measuring points/if necessary, different measurement principles/prevention of false alarms;
- Automatic restart after failure;
- System power supply requirements in inaccessible areas (220V mains power supply; availability of data network, etc.);
- Data pre-processing / compression (no GBs or TBs of raw data);
- Protection against environmental influences vandalism (human and animal);
- Determination of fewer, but more meaningful measurement points per bridge;
- Safe data transmissions; possible data reduction;
- Specification of limit values for warning and alarm stages;
- Appropriate procedures when warning and alarm stages are reached.

There are series of approaches, particularly in the Asian region of installing long-term monitoring systems directly during construction (enormous mechanical stress) and/or as of the

commissioning of a new bridge. Knowledge about the loads to be expected, etc. is necessary here in order to e.g. install sensors at load-relevant points. In particular, experiences gained from the system measurements are the basis for the first long-term measurements installed by DB in order to e.g. drastically reduce the number of measurement points and to determine the appropriate system architecture.
In contrast to short-term measurements (system measurements), long-term measurements also have increased requirements for e.g. the long-term stability of sensor components as well as secure data preprocessing and remote data transmission. The existing test projects very clearly show that there is still some need for expenditure in the sector of R&D as well as for further relevant long-term tests. There is also a need for further developments to be implemented by the measurement technology supplier, while the user (DB) also needs to set out appropriate specifications and standards. The standardization methods that currently exist and those which are in process of being created by the railway can also be guides in other sectors, e.g. in the sector of road bridges.

Acknowledgments

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References

[1] Guideline 805 Tragsicherheit bestehender Eisenbahnbrücken DB Netz AG, August 2008

[2] Geißler, K.: Bewertung und Ertüchtigung stählerner Eisenbahnbrücken. Der Prüfingenieur 38, Issue 04, Page 41 – 53. 2011

[3] Krempels, P., Gommola, G.: MESSTECHNISCHE bestehender Messtechnische Ermittlung Des Systemverhaltens Der Fehmarnsundbrücke Als Grundlage Für Eine Realitätsnahe Bewertung (Bewertungsprofil Nach Richtlinie 805) 6. Symposium "Experimentelle Untersuchungen von Baukonstruktionen" TU Dresden 09.09.2011

[4] Quoos, V.: Dehnungsmessungen unter Verkehrslasten (Straße, Bahn) sowie bei Temperaturgang am Netzwerkbogen und am 5-feldrigen Nebentragwerk der Fehmarnsundbrücke Report 10/38, DB Netz AG, I.NPT 2(F), Magdeburg, December 2010

[5] Gommola, G.: Bridge Evaluation Using Metrology Methods for Short- and Long-Term Measurements Civil Structural Health Monitoring Workshop (CSHM-4)

[6] Geißler, K., Knaack, H.-U., Bolle, G., Marx, S.: Messtechnische Konzepte In Den Regelwerken Von Infrastrukturbetriebern – New Module 805.104 In Guideline 805 7 5. Symposium "Experimentelle Untersuchungen Von Baukonstruktionen" Schriftenreihe Konstruktiver Ingenieurbau Dresden, Issue 18

EVALUATION AND MANAGEMENT OF SEISMIC ENDANGERMENT OF RING ROAD THESSALONIKI

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Abstract

The current paper presents the study that was implemented in the framework of a diploma thesis in the school of Civil Engineering of the Aristotle University of Thessaloniki aiming at evaluating and managing the seismic risk of the internal Ring Road of Thessaloniki in Greece, focusing on the bridges along it. To achieve this, the study includes the following steps: i) the assessment of the seismic hazard of the area, ii) the examination of the structural vulnerability of the bridges and iii) the redirection of traffic in the adjacent urban road network after the occurrence of seismic faults on some of the examined bridges. Towards this direction firstly, some general meanings are presented related to the seismic risk of everyday life activities' networks and the factors which influence them, namely the seismic hazard of the area, the structural vulnerability and the importance of the element under consideration, specializing in road networks. Secondly, the available methods for classification and creation of vulnerability curves of bridges at international and national level are described in detail. The geological and geotechnical aspects of the study area are given and in combination with the available soil simulants, the seismic response of the soil in the area around the bridges for a certain earthquake scenario is calculated. The earthquake scenario is calculated with an average period of reintroduction T_m=475 years and maximum acceleration on rocky background equal to 0.25g. Based on the method FEMA-NIBS (HAZUS), the corresponding vulnerability curves for all the bridges along the inner Ring Road are notified that are likely to experience higher levels of damage in the case of the examined earthquake scenario. With the results presented above, scenarios of traffic rearrangement are presented, in case of a possible blockade of roads due to repair works in two of these bridges. To this end, the best alternative route of the road network in the area is identified and the new distances and timing of such routes are evaluated. Finally, the conclusions drawn from the present thesis work are summarized.

Keywords: vulnerability curves, seismic risk, Ring Road of Thessaloniki, soil stimulants, traffic rearrangement

1 Introduction

Earthquakes are a very common natural phenomenon, particularly in Greece, and affect the operation of networks to a great extent. Despite this, however, these networks often comprise elements lacking seismic design, thus presenting a high probability of failure. In this light, earthquake engineering of utility networks – which aims at the assessment and management of the networks' seismic risk – is a most timely issue and a strategic field of activity under continuous development. The hazards deriving from Greece's high seismicity have not surprisingly raised many concerns. For this reason, it is imperative that seismic design be applied to all infrastructures and elements exposed to the natural phenomenon in order to reduce risk; this is primarily important for existing vulnerable structures. The necessary prerequisite

before taking action is to create and deploy an appropriate methodology for the assessment of seismic losses on the basis of which earthquake scenarios can be applied in order to rank priority policies in terms of preseismic and metaseismic design for the protection of areas. Given the exorbitant cost of applying overall reinforcement to all existing structures, attention has been focused on the logic of selective intervention based on the results of seismic risk assessment studies. Such studies are particularly important and imperative when they relate to utility networks and infrastructures.
The scope of this paper is to assess the seismic risk of bridges along the Thessaloniki Inner Ring Road and the redistribution of traffic to the adjacent urban road network in the event of a failure due to earthquake. This is undoubtedly an effort which due to the nature of the investigation, entails a great degree of uncertainty. Perhaps it would be more plausible to describe the objective as a prediction of the degree of losses on the elements under risk, i.e. whatever can potentially be exposed to the impact of a seismic excitation. As concerns the redistribution of traffic, the objective is to determine the optimum routes in terms of capacity within the congested urban road network of the area under investigation

2 Measurement instruments

In the context of investigating the seismic response of the area along the Thessaloniki Ring Road, two series of one-dimensional seismic ground response analyses were performed for the earthquake scenario with a mean recurrence interval of T_m = 475 years and maximum acceleration on the rocky subsoil equal to 0.25g. The first series of analyses was performed using EERA software, by means of nine (9) ground sections with a thickness range of 2 – 75 m and one (1) seismic excitation (Kozani '95). Respectively, the second series of analyses was performed using STRATA software, by means of seventeen (17) ground sections with a thickness range of 2 – 143 m and five (5) seismic excitations, namely: a) KOZ95-T, b) THE78 D, c) UMB 98 855-Y, d) MONT 79, e) WWT-180, [1].

Table 1 Parameters of seismic excitations used in the one-dimensional ground response analyses

Fid	Name	Earthquake	Country	Date	Focal Depth (km)	Mw	Station Name	Building type	Geology	Epicentral Distance R (km)	PGA (g)
1	855-Y	Umbria-Marche	Italy	5/4/1998	10	4.8	Cubbio-Piene	Free-field	Rock	18	0.235
2	MONT_T	Montenegro	Yugoslavia	15/4/1979	12	6.9	Herceg Novi-O.S.D.Pav.Sch	Free-field	Rock	65	0.256
3	WWT180	N.Palm Springs	USA	8/7/1986	11	6.2	5072 Whitewater Trout Farm	Free-field	Rock	6	0.492
4	Koz95-T	Kozani	Greece	13/5/1995	14	6.5	Prefecture Kozani	Free-field	Rock	17	0.142
5	Thes78_Dec	Thessaloniki	Greece	20/6/1978	6	6.2	THE_6-City	Free-field	Rock	29	0.074

The series of ground sections along the Ring Road is illustrated in Figure 1. As concerns investigation of the bridges' structural vulnerability, the vulnerability curves have been calculated on the basis of the following lognormal distribution function according to FEMA-NIBS 2003:

$$F(DP \geq DP_i | S) = \Phi\left[\frac{1}{\beta tot} \cdot \ln(\frac{S}{Smi})\right] \quad (1)$$

The Bridge Damage Index (BDI) and Link Damage Index (LDI) were then applied (Table 2) to quantify bridge damage – and the respective network served by the bridge–to the most adverse level which would require redirection of traffic along its network, [2].

Figure 1 Spatial arrangement of ground sections

Table 2 BDI (Bridge Damage Index) – LDI (Link Damage Index) and traffic flow capacity

Bridge damage level	BDI (Bridge Damage Index) U.S.A	BDI (Bridge Damage Index) Greece
Low	0.1	0.1
Moderate	0.3	0.4
Extensive	0.75	0.85
Full	1.0	1.0

$$LDI = \sqrt{\sum_{j=1}^{n} (BDI)^2}$$

BDI=the indeks BDI of bridge J for the I network
JI= the numbers of the bridges for the I network

LDI (Link Damage Index)	Capacity traffic flow (U.S.A)	Capacity traffic flow (Greece)
LDI ‹ 0.5	100% (None network damage)	100% (None network damage)
0.5 ‹ LDI ‹ 1.0	75% (Low network damage)	75% (Low network damage)
1.0 ‹ LDI ‹ 1.5	50% (Moderate network damage)	0% (Moderate network damage)
LDI › 1.5	25% (Extensive network damage)	0% (Extensive network damage)

3 Methodology

The analysis results were used to estimate peak ground acceleration (PGA), peak ground velocity (PGV) and peak ground displacement (PGD), as well as spectral acceleration Sa for various frequencies, taking also into consideration the respective standard deviation of each measurement. The geophysical, geological and geotechnical characteristics of the area were used as a basis for correlation of the results to neighbouring locations so as to separate the Ring Road into zones of similar seismic ground response and calculate the seismic design parameters at surface level. The analysis results are provided in tables and diagrams which display the spatial rearrangement of traffic. Concerning the identification of the level of bridge damage on the Ring Road, two separate steps should be performed: classification of bridges into categories and estimation of the pertinent vulnerability curves. Following an extensive review of the available methods for classification and estimation of vulnerability curves– at both a national and international level – the most comprehensive method for the present study found to be: FEMA-NIBS using HAZUS software. Consequently, the related details were compiled for the entirety of bridges along the Inner Ring Road, with special reference to those most likely to present the highest levels of damage according to the earthquake scenario under investigation. Finally, the optimum alternative routes- in terms of capacity and environmental aspect–of the area's under investigation road network were defined and an indicative calculation was performed to derive the new travel distances and times respectively, [3].

4 Results

The above mentioned analyses using EERA and STRATA software led to the results presented in Aggregated Data Tables 3 and 4.

Table 3 Aggregated results using EERA software

Site	E	N	PGAo (g)	PGVo (m(s)	PGDo (m)	T=0.2s (g)	T=0.3s (g)	T=0.4s (g)	T=0.5s (g)	T=0.7s (g)	T=1.0s (g)	T=1.2s (g)	T=1.5s (g)	T=1.7s (g)	T=2.0s (g)	T=2.2s (g)	T=2.5s (g)	T=3.0s (g)
A07			-0.424	0.233	0.015	1.277	1.063	0.996	0.775	0.306	0.205	0.143	0.053	0.038	0.040	0.038	0.039	0.024
A07_16.5			-0.371	-0.186	0.008	1.294	1.271	0.840	0.485	0.180	0.128	0.115	0.040	0.035	0.035	0.032	0.035	0.023
A07_35			-0.359	0.238	0.013	1.092	0.816	0.925	0.687	0.213	0.167	0.128	0.047	0.037	0.038	0.035	0.036	0.023
A07_47			-0.410	0.248	0.014	1.477	0.953	0.998	0.679	0.206	0.162	0.128	0.047	0.039	0.039	0.036	0.035	0.023
A07_63			-0.416	0.243	0.014	1.258	0.992	0.968	0.766	0.287	0.197	0.139	0.050	0.038	0.040	0.037	0.038	0.024
F33			-0.284	-0.009	0.000	0.927	0.618	0.516	0.303	0.133	0.115	0.103	0.040	0.032	0.033	0.027	0.034	0.022
F35			-0.273	-0.005	0.000	0.903	0.609	0.511	0.302	0.133	0.115	0.103	0.040	0.032	0.033	0.027	0.034	0.022
F37			-0.266	-0.004	0.000	0.889	0.603	0.507	0.301	0.132	0.115	0.102	0.040	0.032	0.033	0.027	0.034	0.022
F37_0-2			-0.261	0.000	0.000	0.876	0.597	0.506	0.301	0.132	0.115	0.102	0.040	0.032	0.033	0.027	0.034	0.022
F37_0-5			-0.266	-0.002	0.000	0.889	0.603	0.508	0.301	0.132	0.115	0.103	0.040	0.032	0.033	0.027	0.034	0.022
F37_0-10			-0.283	-0.010	0.000	0.921	0.617	0.515	0.304	0.133	0.116	0.103	0.040	0.032	0.033	0.027	0.034	0.022
F37_0-20			-0.342	-0.046	0.001	1.062	0.670	0.543	0.314	0.137	0.116	0.105	0.040	0.032	0.033	0.028	0.034	0.022
F37_2-2			-0.265	-0.002	0.000	0.887	0.602	0.508	0.301	0.132	0.115	0.103	0.040	0.032	0.033	0.027	0.034	0.022
F37_2-5			-0.269	-0.005	0.000	0.901	0.609	0.511	0.302	0.133	0.115	0.103	0.040	0.032	0.033	0.027	0.034	0.022
F37_2-10			-0.293	-0.015	0.000	0.934	0.620	0.517	0.305	0.134	0.116	0.103	0.040	0.032	0.033	0.027	0.034	0.022
F37_2-20			-0.316	-0.046	0.001	1.065	0.670	0.543	0.315	0.137	0.116	0.105	0.040	0.032	0.033	0.028	0.034	0.022
F37_2-30			-0.298	-0.068	0.002	1.181	0.729	0.575	0.328	0.142	0.118	0.105	0.040	0.033	0.033	0.028	0.034	0.022
F37_2-40			-0.306	-0.108	0.004	1.145	0.786	0.615	0.355	0.150	0.121	0.107	0.040	0.033	0.034	0.029	0.034	0.022
J39			-0.397	-0.052	0.001	1.117	0.688	0.553	0.317	0.138	0.116	0.106	0.041	0.032	0.033	0.028	0.034	0.022
N01			-0.302	0.161	0.018	0.771	0.876	0.000	0.621	0.240	0.258	0.211	0.077	0.060	0.051	0.040	0.045	0.029
N40			-0.389	-0.187	0.009	1.203	1.002	0.799	0.469	0.173	0.129	0.116	0.041	0.034	0.035	0.032	0.034	0.023
N40_15			-0.446	-0.098	0.002	1.601	0.814	0.610	0.341	0.145	0.119	0.109	0.042	0.033	0.033	0.028	0.034	0.022
N40_30			-0.408	-0.164	0.006	1.657	1.061	0.729	0.407	0.166	0.126	0.109	0.040	0.034	0.034	0.031	0.034	0.022
P40			-0.390	0.245	0.013	1.621	0.937	0.901	0.607	0.212	0.159	0.127	0.046	0.037	0.038	0.035	0.036	0.023
P40_37			-0.438	-0.211	0.009	1.410	1.263	0.893	0.514	0.185	0.132	0.117	0.041	0.036	0.036	0.033	0.035	0.023
R39			-0.298	-0.174	0.013	1.213	0.861	0.729	0.445	0.235	0.210	0.153	0.056	0.042	0.040	0.033	0.039	0.024

Table 4 Aggregated results using STRATA software

Site	East	North	PGAo (g)		PGVo (cm/s)		PGDo (cm)	
			Mean value m	St. Deviation s	Mean value m	St. Deviation s	Mean value m	St. Deviation s
A05_855			0.329	0.067	24612.989	0.259	3300.104	0.306
A07			0.388	0.088	24325.618	0.267	3254.466	0.322
A07_16.5			0.398	0.054	20674.622	0.153	2755.649	0.331
A07_35			0.360	0.106	22118.123	0.233	3064.301	0.337
A07_47			0.349	0.153	3164.412	0.344	3164.412	0.344
A07_63			0.336	0.124	23823.828	0.278	3216.769	0.332
B04_855			0.376	0.135	23666.364	0.277	3197.397	0.346
C03_855			0.362	0.111	22854.721	0.247	3167.019	0.326
D02_855			0.347	0.104	23173.655	0.237	3206.980	0.321
F01_855			0.355	0.074	24590.776	0.219	3324.345	0.313
F33			0.296	0.161	13603.632	0.170	2568.896	0.300
F35			0.280	0.128	13492.327	0.174	2576.881	0.307
F37			0.273	0.103	13472.810	0.174	2582.420	0.310
F37_0-2			0.250	0.025	13436.348	0.176	2599.117	0.319
F37_0-5			0.266	0.069	13459.748	0.175	2587.262	0.312
F37_0-10			0.283	0.116	13542.723	0.172	2566.440	0.301
F37_0-20			0.311	0.079	14002.724	0.167	2572.493	0.296
F37_2-2			0.262	0.047	13464.719	0.174	2591.908	0.314
F37_2-5			0.277	0.120	13481.299	0.174	2577.364	0.307
F37_2-10			0.288	0.072	13635.394	0.171	2562.730	0.298
F37_2-20			0.320	0.098	14137.672	0.165	2578.705	0.297
F37_2-30			0.324	0.070	14657.435	0.166	2613.781	0.307
F37_2-40			0.316	0.050	15433.829	0.178	2634.071	0.315
H01_855			0.276	0.065	23011.567	0.273	3642.477	0.274
J01_855			0.340	0.099	4064.345	0.305	4064.345	0.305
J39			0.373	0.137	14706.858	0.135	2591.745	0.293
L01_855			0.284	0.081	24658.367	0.294	3926.527	0.286
N01			0.294	0.060	20930.409	0.231	3637.207	0.274
N40			0.383	0.026	19405.501	0.164	2798.368	0.327
N40_15			0.447	0.057	16157.415	0.155	2649.927	0.301
N40_30			0.435	0.095	18415.331	0.136	2718.868	0.321
P40			0.367	0.144	21745.482	0.265	3097.862	0.337
P40_37			0.403	0.053	21112.300	0.157	2867.836	0.319
R39			0.279	0.058	20351.713	0.280	3395.375	0.267

The procedure followed for the seismic risk assessment of bridges on the Thessaloniki Inner Ring Road produced the results summarized in Table 5.

Table 5 Aggregated results of seismic risk assessment

FID	BRIDGE/PLACE	YEAR	SPANS	TYPE	SUPPORT	CONTINUOUS	Sa (1.0 s)	Sa (0.3 s)	P [>=Low]	P [>=Moderate]	P [>=Extensive]	P [>=Collapse]	P [=None]	P [>=Low]	P [>=Moderate]	P [>=Extensive]	P [>=Collapse]	max P[=DS]	Expected D.L.[=]	Expected D.L.[>=]
0	Underpass to cementeries	1990	1	compact slab reinforced concrete	simple mounting	YES	0.27	1.20	0.04	0.02	0.01	0.00	0.96	0.01	0.01	0.01	0.00	0.96	1	1
1	GSC to junction K5 (Hospital Papageorgiou)	2003	1	compact slab reinforced concrete	monolithic	YES	0.28	1.36	0.04	0.02	0.01	0.00	0.96	0.02	0.01	0.01	0.00	0.96	1	1
2	Retziki Street GSC to junction K6	1988	1	beam prestressing concrete	simple mounting	YES	0.19	0.67	0.01	0.00	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.99	1	1
3	GSC to junction K7, E ptapirgiou Area	1987	1	concrete box intersection	simple mounting	YES	0.19	0.65	0.01	0.00	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.99	1	1
4	Viaduct (km position. 21+662.07)	1984	1	beam prestressing concrete	simple mounting	YES	0.19	0.65	0.01	0.00	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.99	1	1
5	Viaduct (km position. 22+576.66)	1984	1	beam prestressing concrete	simple mounting	YES	0.19	0.67	0.01	0.00	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.99	1	1
6	GSC to Saint Paul Area	1988	1	beam prestressing concrete	simple mounting	YES	0.19	0.65	0.01	0.00	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.99	1	1
7	GSC A-B section to junction K8 (km position 1+079.58)	2002	7	concrete box intersection	monolithic	YES	0.19	0.65	0.02	0.00	0.00	0.00	0.98	0.02	0.00	0.00	0.00	0.98	1	1
8	Toumpa Area GSC to junction K9	2002	7	concrete box intersection	monolithic	YES	0.19	0.67	0.03	0.00	0.00	0.00	0.97	0.02	0.00	0.00	0.00	0.97	1	1
9	Bridge stream Krioneri	1992	1	compact slab reinforced concrete	monolithic	YES	0.19	0.67	0.01	0.00	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.99	1	1
10	GSC to junction K10 East Ring Road	1994	1	beam prestressing concrete	simple mounting	YES	0.19	0.67	0.01	0.00	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.99	1	1
11	Underpass Ring Road (km position 26+282.00)	1990	1	compact slab reinforced concrete	simple mounting	YES	0.42	0.80	0.14	0.08	0.04	0.01	0.86	0.06	0.04	0.03	0.01	0.86	1	1
12	Pilaia-Panorama street GSC to junction K11	1990	2	slab with cydically interstices concrete	simple mounting	YES	0.21	1.15.	0.04	0.00	0.00	0.00	0.96	0.03	0.00	0.00	0.00	0.96	1	1
13	N.Diagonios Grade Separated Crossing (GSC) to junction K12	1992	3	concrete box intersection	simple mounting	YES	0.36	0.92	0.53	0.30	0.20	0.07	0.47	0.22	0.11	0.13	0.07	0.47	1	2
14	Carriageway A of N.Diagonios GSC to junction K12	1992	3	concrete box intersection	simple mounting	YES	0.36	0.92	0.53	0.30	0.20	0.07	0.047	0.22	0.11	0.13	0.07	0.47	1	2
15	Underpass Section A to junction K12	1992	1	beam prestressing concrete	simple mounting	YES	0.36	0.92	0.09	0.05	0.03	0.01	0.91	0.04	0.03	0.02	0.01	0.91	1	1
16	Monastiriou Street GSC to junction K17	1978	7	beam prestressing concrete	form type GERBER	NO	0.46	0.95	0.85	0.66	0.49	0.23	0.15	0.19	0.16	0.27	0.23	0.27	4	3
17	Lagkadas Road Interchange (I/C) to junction K18	1985	10	beam prestressing concrete	simple mounting	NO	0.33	1.03	0.68	0.49	0.33	0.12	0.32	0.19	0.16	0.21	0.12	0.32	1	2

By analysing the values of the table of results one can conclude that the expected performance of almost all the bridges under a possible seismic excitation is deemed as satisfactory. Nevertheless some of these call for special attention, due to their expected level of damage. Specifically, these bridges are:

1 N. Diagonios Grade Separated Crossing (GSC) to junction K12;
2 Carriageway A of N. Diagonios GSC to junction K12;
3 Monastiriou Street GSC to junction K17;
4 Lagkadas Road Interchange (I/C) to junction K18.

The purpose of the study was to identify the anticipated seismic damage to Thessaloniki bridges; to this end, only two bridges were examined, those being the ones presenting the highest levels of damage for the basic earthquake scenario with a mean recurrence interval of T_m=475 years, as shown in Figure 2. Specifically, the GSC of Monastiriou Street at junction K17 was assessed to sustain extensive damage (BDI equal to 0.85), while lower damage is expected at the I/C of Lagkadas Road at junction K18 (BDI equal to 0.1).

Figure 2 Visual representation of locations and bridge damage level on the Inner Ring Road

Using the BDI and LDI indices:

$$LDI = \sqrt{0.85^2 + 0.1^2} = 0.86 \quad (2)$$

Consequently, as concerns the local bridge network under investigation, the level of damage is expected to be low, since the LDI index was found equal to 0.86 and according to Table the traffic flow capacity will be reduced to 75% of its total. This data was the basis for proposing the respective traffic rearrangement scenarios in the event of roads being blocked due to repairs on the two bridges that were examined extensively. Thus at Lagkadas Road I/C there is zero increase in the new length of the alternative route, owing to the existence of a side road of satisfactory traffic capacity, whereas at Monastiriou Street GSC there is a moderate increase in the new travel distance in the eastward direction (1.90 km instead of 1.30 km) and a great increase in the westward direction (5.40 km instead of 1.30 km). Moreover, the new travel distances, in conjunction with reduced traveling speeds due to the poor geometric characteristics of the alternative road segments (45 km/h as opposed to 90 km/h) lead to two-fold (0.73 min instead of 0.37 min, Lagkadas Road I/C) or even 9-fold (7.2 min instead of 0.87 min, Monastiriou Street GSC in westward direction) travel times respectively.

5 Conclusions

In conclusion, the results of the present investigation contribute towards three interesting topics, i.e. the assessment of the seismic hazard of the area, the examination of the structural vulnerability of the bridges and the impact of the redirection of traffic on the adjacent urban road network in case of bridges failure. Concerning the assessment of the seismic hazard of the area it was found that the maximum PGA with an average value of 0.447 g at ground surface is expected on the eastern part of Thessaloniki, within the boundaries of the Municipality of Kalamaria. As concerns the study of structural vulnerability of bridges along the Inner Ring Road, these are as a whole in satisfactory condition and are not expected to sustain serious damage in a potential seismic excitation under the examined earthquake scenario, with minor exceptions. Finally, concerning the impact of the redirection of traffic on the adjacent urban road network the main finding is that the lack of a sufficient transportation infrastructure in the Urban Agglomeration of Thessaloniki does not provide with alternative routes of appropriate capacity in case of the closure of critical elements-as the case of bridges- leading thus to increase in travel time and cost and generally to users' inconvenience during the period of repair works.

References

[1] Kakderi, K., Argiroudis, S., Fotopoulou, S. & Pitilakis, K.: Earthquake scenarios and vulnerability of utility networks and infrastructures in the town of Grevena, 6th Pan-Hellenic Conference on Geotechnical and Geoenvironmental Engineering, Technical Chamber of Greece, Volos 29 September – 1 October 2010.

[2] SRM – LIFE Research Programme, Development of an Integrated Methodology for the Seismic Vulnerability Assessment of Utility Networks, Infrastructures and Strategically Important Buildings for Seismic Risk Management in Urban Agglomerations. Application in the Urban Agglomeration of Thessaloniki, 2003 – 2007, Aristotle University of Thessaloniki – Dept. of Civil Engineering, Thessaloniki.

[3] Pitsiava, M. & Argiroudis, S.: Seismic Risk Assessment of road & rail networks, SRM-LIFE Workshop, Thessaloniki, 29/5/2008

[4] Pitilakis, K., Anastasiadis, A., Argiroudis, S., Kakderi, K. & Alexoudi, M.: Vulnerability estimation and seismic risk management of utility networks, infrastructures and critical services, application in metropolitan Thessaloniki, 3rd Pan-Hellenic Conference on Earthquake Engineering and Technical Seismology, Athens 5 – 7 November 2008.

[5] Ioannis Ph. Moschonas: Concrete Bridge Seismic Vulnerability Analysis, doctoral thesis, 2010.

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MOVING LOAD EFFECT ON BRIDGES

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Abstract

Vehicle – bridge dynamic interaction represents the actual problem which is solved on many work places. Within the solution of the task the numerical methods are applied mainly. The Finite Element Method is the best-known and widely used. Several commercial systems works with the algorithms based on the FEM principles, for example program system ANSYS. The submitted article is dedicated to the numerical modelling of vehicle – bridge interaction problem in the environment of this system. It compares the numerical accuracy of FEM with other numerical method and illustrates the influence of the speed of vehicle motion on the bridge mid-span deflection.

Keywords: ANSYS, finite element method, vibration, dynamic analysis, random profile

1 Introduction

Having better possibilities to solve interaction between vehicles and bridge structures can extend knowledge to better understand influence of moving vehicles on bridge structures. The choice of the right model, which represents the moving vehicle with appropriate mathematical formulation, is one of the most important parts [5]. Simple model that describes only a part of dynamic characteristics of the real models are usually preferred. “The fourth-part model” and “the half -part model” are widely used when only the analysis for the plane model of the bridge is required [3]. However, they can be also used for tree-dimensional models of bridges but they cannot describe it properly. So the results of the solution have to be carefully evaluated and interpreted. Discrete models of vehicle with finite degrees of freedom make solution easier from the mathematical point of view. Then partial differential equations change to the differential equations. The article describes the way how to create the right numerical model of the vehicle and also how to create interaction with numerical model of the bridge by using the software ANSYS [1], [6]. For the model of vehicle “the half-part model” is used, created by using spring elements and mass elements. They describe stiffness, damping and mass characteristics of the vehicle. The bridge is modelled by using planar beam elements. Interaction between vehicle and bridge is created by using contact elements node to surface. This task belongs among nonstacionary dynamic actions, which is governed by the following equation:

$$[M]\cdot\{u(t)\}+[C]\cdot\{u(t)\}+[K]\cdot\{u(t)\}=\{F(t)\} \tag{1}$$

One of the possibilities how to solve the previous equation is using numerical solution. Program ANSYS offers many techniques for that purpose. We decided to use Newmark’s method that is widely used to do dynamic numerical simulations. This method is also called implicit because the solution at time t + Δt is not explicitly determined by the state at time t. The relation between displacement, velocity and acceleration is governed by the following equations:

$$\begin{aligned} \{u_{t+\Delta t}\} &= \{u_t\} + \left[(1-\delta)\cdot\{u_t\} + \delta\cdot\{u_{t+Dt}\}\right]\cdot\Delta t \\ \{u_{t+\Delta t}\} &= \{u_t\} + \{u_t\cdot\Delta t\} + \left[(1/2-\alpha)\cdot\{u_t\} + \alpha\cdot\{u_{t+\Delta t}\}\right]\cdot\Delta t^2 \end{aligned} \tag{2}$$

As a result, the time is also discretized and the solution is given in a form of the functional values for all defined geometrical points in each time steps. The different value for the time step affects the quality of the obtained results. So it is very important to choose the right value of the time step.

2 The numerical model of the vehicle

For the purpose of the simulation "the half -part model" was used, which is based on the lorry T815 Fig. 1. Linear damping characteristics and viscous damping are assumed for spring elements that are used to connect mass elements. The dynamic characteristics such as natural frequencies were solved to evaluate the quality of the model. They were compared with natural frequencies gained by experimental measurement [7].

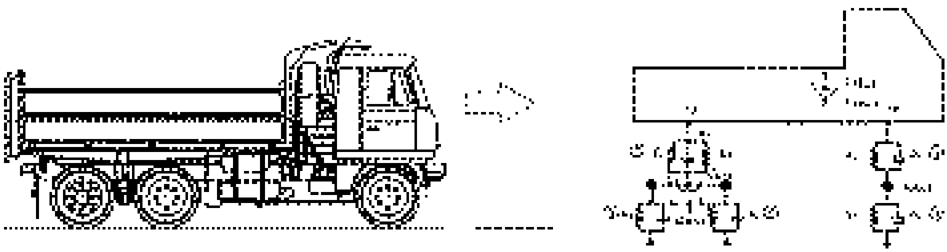


Figure 1 The numerical model of the lorry T815

It is possible to define the main characteristics of the discrete model by three diagonal matrices. They contain the mass, stiffness and damping characteristics.

Diagonal mass matrix

$$\{m\}_D = \{m_1, I_{y1}, m_2, m_3, I_{y3}\}_D = \{17400, 62298, 910, 2140, 932\}_D \quad [kg,\ kg\cdot m^2]$$

Diagonal stiffness matrix

$$\{k_i\}_D = \{k_1, k_2, k_3, k_4, k_5\}_D = \{247686, 1869724, 3242424, 5095000, 5095000\}_D \quad [N/m]$$

Diagonal damping matrix

$$\{b_i\}_D = \{b_1, b_2, b_3, b_4, b_5\}_D = \{\ 19228, 260197, 2746, 5494, 5494\}_D \quad [kg/s]$$

3 The numerical model of the bridge

The goal of the analysis is to analyse the action of moving vehicle on the structure of the road bridge situated between two villages Varín and Mojš in the Slovak republic. The whole length of the bridge is 87 m which is divided into three parts. Each part acts independently as a simple supported beam. The main girders are prefabricated prestressed concrete elements and their length is 29 m. The cross-section of the bridge contains eight girders placed at distance 1.44 – 1.45 m. The shapes of the girders and the layers of pavement are showed on the Fig. 2. The Young's modulus is 3.85 e10 N/m^2 and the weight intensity is 19680 kg/m. Both values are defined with the respect of having similar dynamic characteristic than those the real bridge structure has.

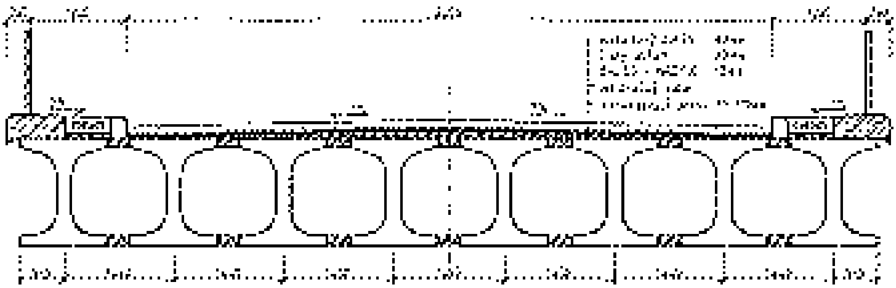
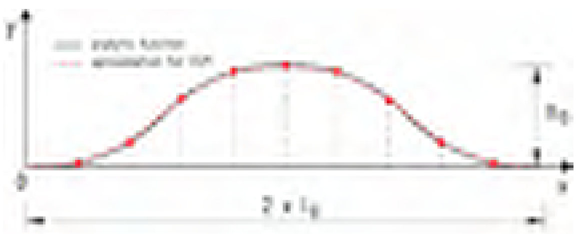


Figure 2 The cross-section of the bridge situated between two villages Varín – Mojš in the Slovak republic

The analysis is focused on the middle span of the bridge that is modelled using beam elements BEAM3. The boundary conditions are similar to a simple supported beam. Rigid surfaces are defined before and after the model of the bridge and their purpose are to start and finish the movement of the vehicle. Thanks to this, the vehicle moves over the bridge with a constant speed. An obstacle is defined on the surface of the bridge. It is positioned in the middle of the span and its shape is very similar to the obstacle used for the dynamic test of bridges Fig. 3. The main reason for defining of this roughness on the surface of the pavement is to have results which could be compared with results of experimental analysis.



Function that defines the shape of the obstacle

$$h(x) = h_0.\left(1 - \cos\left(2\pi x / 2l_0\right)\right) / 2$$

where:

h_0 = 0,06m ,2. l_0=0,940 m

Figure 3 The shape of the obstacle

The simplified model of the bridge and vehicle at the beginning of the numerical simulation is showed below Fig. 4. The static analysis begins to bend the bridge as an effect of self-weight without causing vibrations. Then the type of analysis is changed to dynamic analysis. The inertial forces are active for the rest of the numerical simulation.

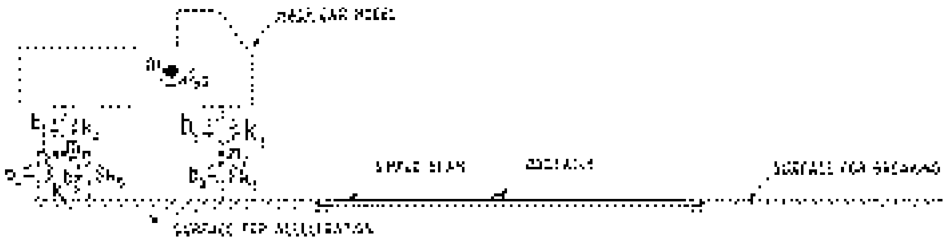


Figure 4 The numerical model of the vehicle and the bridge

4 Analysis of the results

The displacement in the middle of the beam is used to analyse the influence of the obstacle situated on the surface of the bridge. The first solution was done without the obstacle, thus the smooth surface was assumed. The results were nearly the same as it is expected for the static analysis. The displacement changes only by changing the position of the vehicle. The influence of inertial forces is very small. The next solution was done with the defined obstacle in the middle of the bridge. As a result, the displacement changes in time and the maximal value increased. The Fig. 5 shows how it influences the displacement.

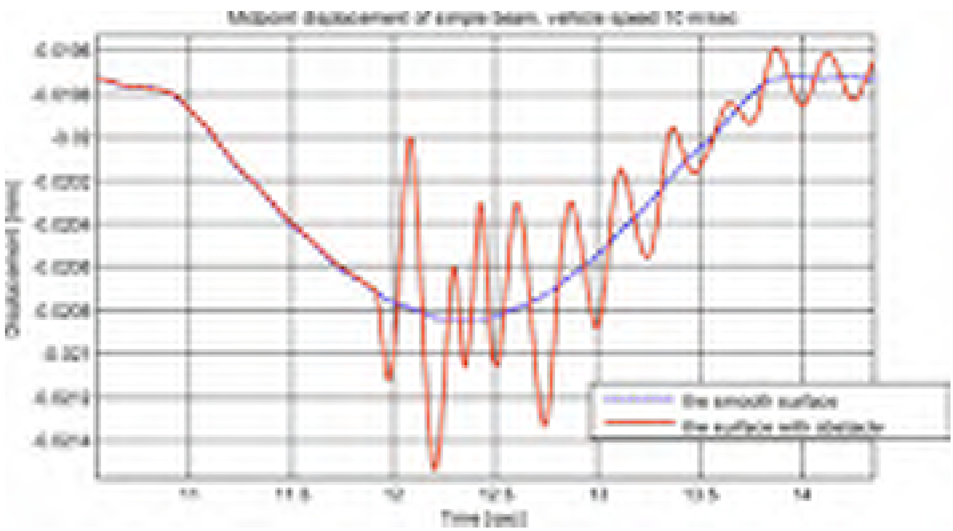


Figure 5 The influence of the obstacle on the displacement in the middle of the span

It was also valuable for this kind of solution to know how the different values of damping can change the displacement. So another analysis was done where the results for model without damping and with damping are compared. Damping was defined by two numbers α =0.03 and β =0.002. These values are derived from the results of experimental analysis. The change of displacement in time is showed in figure 6. It is obvious that the maximal value of the displacement did not change significantly, but the vibration disappears faster.

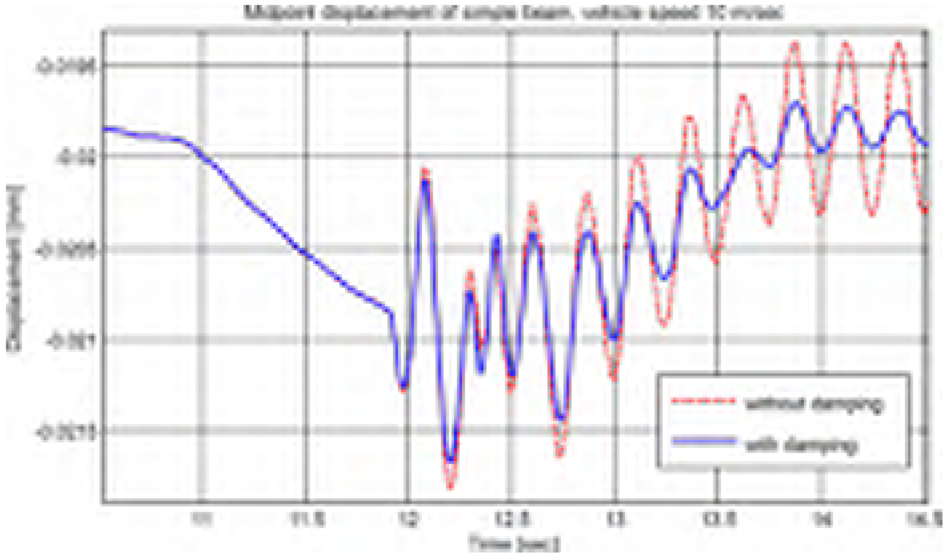


Figure 6 The influence of damping on the displacement in the middle of the span

5 Conclusion

The numerical simulation of the moving vehicle acting on the bridge structure needs many simplifications if we want to use it for solving some practical issues. That is even more important for solving that simulation with prescribed roughness of the road on the bridge structure. For that reason the simple plane model of vehicle and bridge was chosen. The comparison between simulation with smooth surface on the bridge and simulation with obstacle in the middle of the span shows significant changes in the values of displacement. For the smooth surface the results are comparable with static analysis. On the other hand the displacement with obstacle on the bridge increases. There is also important to define right value of damping especially if fatigue phenomenon is analysed.

References

[1] ANSYS, Inc.: ANSYS 8.0 Documentation

[2] STN ISO 8608 Mechanical vibration. Profiles of pavements. Aquisition of measured quantities (Mechanické kmitanie, profily povrchu cesty. Zaznamenávanie nameraných údajov), SÚTN, Bratislava, 2000.

[3] Daniel, Ľ.: Vehicle-road interaction (Interakcia v sústave vozidlo – jazdná dráha), Práca ŠVOČ, Svf, ŽU, Žilina, 2012.

[4] Melcer, J.: Dynamic analysis of the road bridges (Dynamické výpočty mostov na pozemných komunikáciách). EDIS, Žilinská univerzita v Žiline, 1997.

[5] Frýba, L.: Vibration of Solids and Structures Under Moving Loads. ACADEMIA, Praha, Nordhoff International Publishing, Groningen, 1972.

[6] Ellis, H. Dill: The Finite Element Method for Mechanics of Solids with ANSYS Applications, CRC Press, 2012.

[7] Jimin He, Zhi-Fang Fu: Modal Analysis, Butterworth-Heinemann, 2001.

REHABILITATION OF STEEL RAILWAY BRIDGES BY IMPLEMENTATION OF UHPFRC DECK

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Abstract

Nowadays on the existing railway infrastructures many bridges can be found that have been built more than 50 years ago, and which were not designed for current loads and high speed trains. These are mainly bridges from hot rolled steel or cast iron and connected with rivets. The main idea of strengthening existing steel bridges is considering the possibility of adding load bearing deck above the main girders without replacing them. Converting alone metal section to composite cross-section raises the centre of gravity so that section can handle additional loads. In addition, the concrete deck may stiffen upper steel flange and thus eliminates the problem of stability of compressed part of the cross-section. In this paper the research on innovative rehabilitation method for the existing steel bridges is presented. The research has been performed within FP7 SMART RAIL project, and is based on the case study of rehabilitation project of the small non-ballasted steel bridge. The selected bridge ("Buna" bridge in Croatia) was built in 1893, with the first reconstruction in 1953. Since the steel structure of the old Buna Bridge had to be completely replaced, the bridge was dismantled and transported to the laboratory for the experimental assessment and development of the new rehabilitation method. The new design is based on the implementation of the prefabricated ultra-high performance fibre reinforced concrete (UHPFRC) deck. It is expected that this strengthened cross-section will be able to withstand the increased load, as required by contemporary regulations.

Keywords: steel bridge, railways, rehabilitation, ultra-high performance concrete

1 Introduction

Nowadays on the existing railway infrastructures many bridges can be found that have been built more than 50 years ago, and which were not designed for current loads and high speed trains. These are mainly bridges from hot rolled steel or cast iron and mainly connected with rivets. By economic and environmental reasons, it would be a great benefit to extend the service life of these bridges, instead of demolishing or reconstruction. The main idea of strengthening existing steel bridges is considering the possibility of adding load bearing deck above the main girders without replacing them, [1-3]. Converting alone metal section to composite cross-section raises the centre of gravity so that new composite cross-section can carry additional loads. In addition, the concrete deck stiffen upper steel flange and thus eliminates the problem of stability of compressed part of the cross-section.
In this paper the research on innovative rehabilitation method for the existing steel bridges is presented. The research has been performed within FP7 SMART RAIL project, and is based on the case study of rehabilitation project of the non-ballasted steel bridge, [4].

2 Case study: Buna bridge

2.1 Description of the Buna bridge

Buna Bridge in Croatia was built in 1893, with the first reconstruction in 1953 which held up to date. Bridge spans the creek Buna and is located on the Zagreb-Sisak railway line. The bridge is about 9 meters long and 0.9 m high. Cross-section consists of two main girders made of hot rolled steel plates joined with rivets and represents a typical beam construction from the time in which it was created. Main girders are connected by horizontal and vertical grids for stiffening. Because of its location near Zagreb and handling weight of only 8.0 tons, this non-ballasted bridge was a perfect choice for transportation in the testing laboratory.

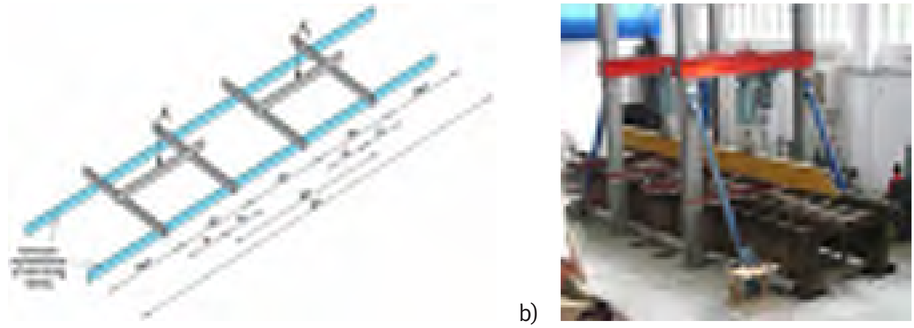
Figure 1 Location of Buna bridge

2.2 Condition assessment of existing bridge in the laboratory

To obtain results for comparison, it was decided that the Buna bridge will be tested before and after strengthening at the Laboratory for Structures at Institute IGH in Zagreb.

Figure 2 Bridge transported into laboratory

It is assumed that the bridge will withstand the load according to EN 1991-2 using Load Model 71 [5]. It is four axle load of 250 kN each at a distance of 1.6 m. The characteristic values are multiplied by a factor α on lines carrying rail traffic which is heavier or lighter than normal rail traffic. The value 1.33 is normally recommended on lines for freight traffic and international lines [6]. Finally, it is necessary to increase the load by dynamic factor for standard maintained track Φ_3. For simply supported girders dynamic factor Φ_3=1.527 for determinant length L_Φ=8.47 m is used so the total force load amounts 1530 kN.



a) b)

Figure 3 a) Schematic representation and b) Load distribution during testing.

During testing following parameters were measured: applied forces; vertical deflection of each girder on elastomeric supports; vertical deflection in the middle of the span; and stresses on upper and lower flange of each girder in the middle of the span. The results are given in diagrams in Figure 4 and Figure 5 and in Table 1. Equipment used for testing was following:

- two load cells – capacity each 1000 kN;
- two hydraulic pistons – capacity each 1000 kN;
- six linear variable differential transformers (LVDT);
- four strain gauges – 120Ω, base length 10 mm;
- National Instruments PXI and SCXI acquisition device.

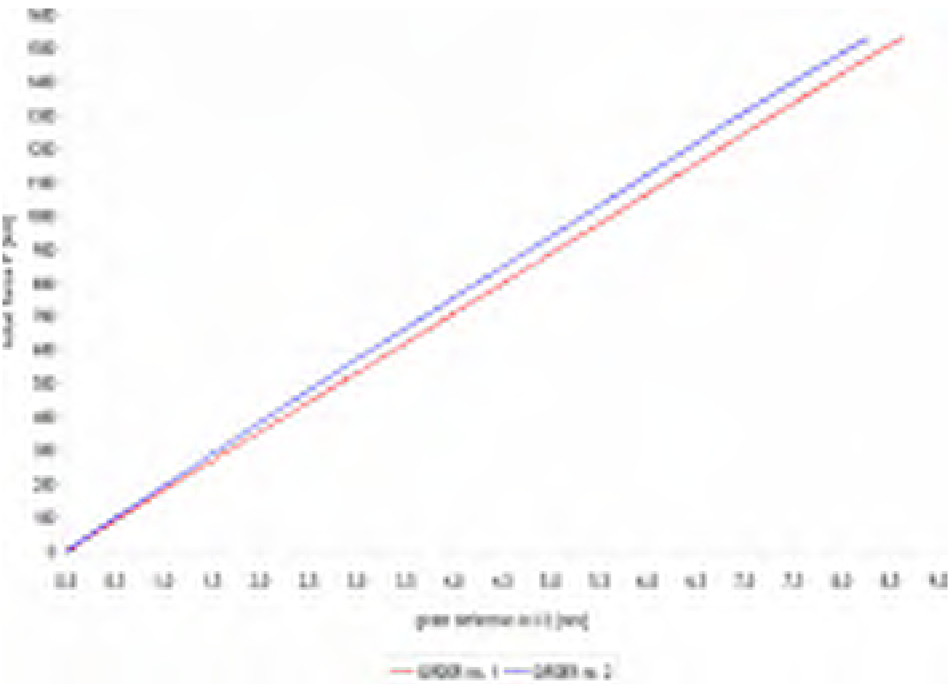


Figure 4 Diagram of girder deflection in l/2 during testing

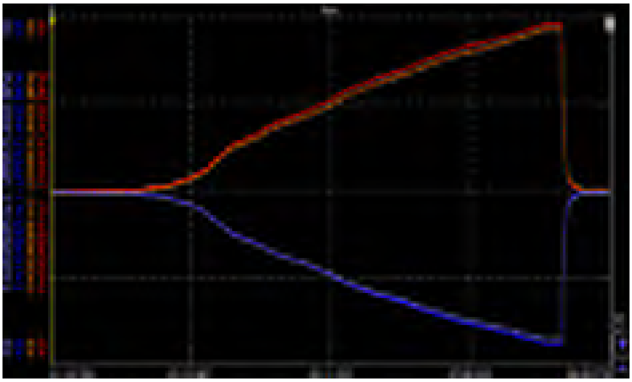


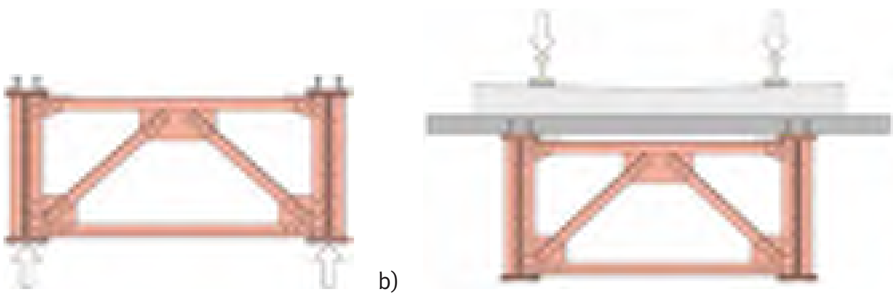
Figure 5 Diagram of stresses during testing

Table 1 Laboratory testing results

Force [kN]			Displacement in l/2 [mm]		Flange stress [MPa]			
F_1	F_2	total (F_1+F_2)	girder no. 1	girder no.2	girder no. 1		girder no. 2	
					upper	lower	upper	lower
756.53	772.52	1529.05	8.64	8.28	-89	101	-87	98

2.3 Design concept of the bridge strengthening

Main idea is to convert alone metal section to composite cross-section. The composite cross-section will increase load bearing capacity and reduce the stress range of live load. The dynamic effects of live loading will also be reduced and concentrated load effects on non-ballasted steel structure would be diminished. Steel structure will be supported during casting in order to increase the bearing capacity of the bridge structure, as schematically presented in Figure 6a, because composite cross-section already carries its own weight and all additional loads.



a) b)

Figure 6 Cross-section a) before and b) after the completion

The new design will be based on the implementation of the cast in situ ultra-high performance fibre reinforced concrete (UHPFRC) deck, as it can be seen in Figure 6b, because of the exceptional mechanical and durability properties of this material.

2.4 Laboratory testing after rehabilitation

Besides testing with the same load arrangement as before rehabilitation, it is planned to test the bridge after rehabilitation with load arrangement and magnitude according to Load

Model SW/2 which represents the static effect of vertical loading due to heavy rail traffic (it is a continuous load of 150kN/m). At the moment of finalizing this paper steel headed dowels are being prepared for welding to the main steel girders, after which ultra-high performance fibre reinforced concrete (UHPFRC) deck will be cast in-situ. Current status of rehabilitation work on the bridge structure can be seen in Figure 7. It is also planned to carry out fatigue testing on bridge after rehabilitation, using cyclic loading with a frequency and duration yet to be contemplated. It is expected that this strengthened cross-section will be able to withstand the increased load, as required by contemporary regulations.

Figure 7 Bridge in process of rehabilitation

3 UHPFRC based rehabilitation methods

Ultra-High Performance Fibre Reinforced Concrete (UHPFRC) is an exceptional cmentitious material characterized by a unique combination of extremely low permeability, high strength and deformation capacity (tensile strain hardening, Figure 8). Extensive R&D works and applications over the last 10 years have demonstrated that cast on site UHPFRC is a fast, efficient and price competitive repair and rehabilitation method for existing structures. UHPFRC provides the structural engineer with a unique combination of extremely low permeability, high strength and tensile strain hardening material. UHPFRC is perfectly suited to the rehabilitation of reinforced concrete structures in critical zones subjected to an aggressive environment and to significant mechanical stresses, to provide a long-term durability and thus avoid multiple interventions on structures during their service life, following maintenance strategy "A" as presented in Figure 9.

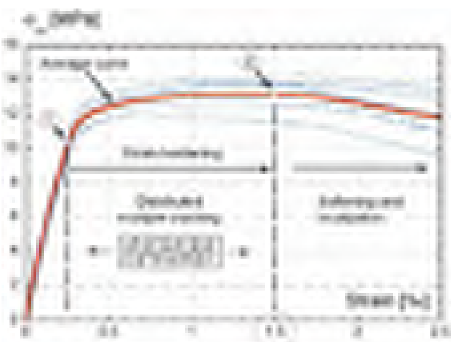


Figure 8 Tensile response of UHPFRC (results from 5 dog bone specimens and average curve, after [7])

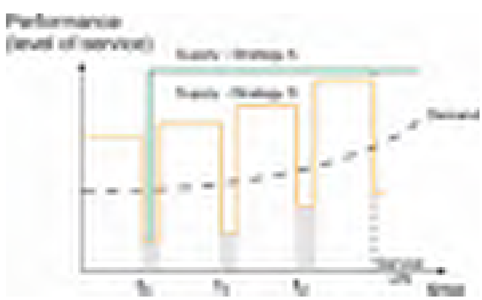


Figure 9 Maintenance strategy of preventing multiple interventions on structures during their service life, [7]

Recent real on-site applications have that UHPFRC can, apart from non-bearing rehabilitations (Log Čezsoški, Ligtening tower), successfully be applied for strengthening of different structures (Strengthening of a 50 year-old reinforced concrete floor of a fire brigade building in Geneva in view of heavier future fire engines(2007), rehabilitation of a 28.5 m span bridge deck of bridge “Dalvazza” (2008)). By introducing an original concept of ECO-UHPFRC with a high dosage of mineral addition, a low clinker content, and a majority of local components, which was within the FP7 research project ARCHES successfully applied for the rehabilitation of a bridge Log Cezsoski in Slovenia, it has been shown, that UHPFRC based rehabilitation methods can also be more sustainable than traditional ones [8]. Based on 10-year on-site experiences a FP7 research project SmartRail has been launched, which goal, among others, was to transfer, apply and prove the UHPFRC based rehabilitation techniques for the rehabilitation of old steel bridges. The advantages of UHPFRC particularly valuable for rehabilitation of railway bridges are high strength and ductility, low added dead load, low added thicknesses, i.e. change in the track vertical alignment, extreme durability. Based on an extensive research work a UHPFRC composition whose main characteristics are listed in Table 2 was chosen to be applied on the old steel railway bridge Buna, Croatia for one-to-one in-lab investigations. The test performed and the results are presented in the next chapter.

Table 2 Main characteristics of UHPFRC used in the investigation

Cement content	540 kg/m^3
Mineral addition	800 kg/m^3
Steel fibres	450 kg/m^3
Superplasticizer	0,9 % on cement
Aggregate 0/2	570 kg/m^3
Compressive strength @ 28 days	150 MPa
Bending strength @ 28 days	35 MPa
Air permeability (Torrent method)	0,01E-16 m^2
Restrain shrinkage	No cracking

4 Conclusion

The majority of the remaining old steel railway bridges will not pass assessment using modern codes of practice. This is because of the extra loading associated with the installation ballasted deck loading, combined with on-going corrosion and fatigue cycles. This is particularly apparent in the deck area due to the lack of drainage and the dynamic impact loadings particularly on non-ballasted bridges. The bridges as a whole may have the capacity to resist the current loading as factored using modern risk and probability factors, however with the desired loading of modern design codes, the requirements of extra deck loading and a desirable extended life of the structure, the bridge will generally fail assessment. This normally occurs because of lack of capacity in the deck with the main load bearing structural elements and other elements either failing by a small percentage or just being sufficient. The UHPFRC composite deck system has been shown to be of significant benefit to this bridge structure. The composite deck has increased the decks capacity and reduced the stress range of live load stress. It has reduced the dynamic factor and concentrated load effects on the previous steel structure.

Acknowledgement

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References

[1] Caroll, M.: An Investigation into Concrete Composite Strengthening of a Continuous Wrought Iron Railway Bridge using Precast Concrete Deck Slabs Bridge, University of Surrey, MSc Thesis, 2010.

[2] Network Rail, Railway Bridges Today and Tomorrow, Proceedings of the conference, UK, 2006.

[3] Olofsson, I., Elfgren, L., Bell, B., Paulsson, B., Niederleithinger, E., Sandager, J., Jensen, Feltrin G., Täljsten, B., Cremona, C., Kiviluoma, R., Bien, J.: Assessment of European railway bridges for future traffic demands and longer lives – EC project “Sustainable Bridges”, Structure and Infrastructure Engineering, Vol. 1, No. 2, June 2005, pp. 93 – 100.

[4] SMART Rail project, FP 7 project, http://smartrail.fehrl.org/

[5] EN 1991 – 2 Eurocode 1 : Actions on structures – Part 2: Traffic Loads On Bridges

[6] UIC CODE 702, 2003

[7] Denarié, E. & Brühwiler, E.:, Structural rehabilitations with Ultra High Performance Fibre Reinforced Concretes, International Journal for Restoration of Buildings and Monuments, Aedificatio, Vol. 12, No. 5 and 6, pp. 453-467, 2006.

[8] Habert, G., Denarié, E., Šajna, A., Rossi, P.: Lowering the global warming impact of bridge rehabilitations by using Ultra High Performance Fibre Reinforced Concretes, Cement & Concrete Composites 38 (2013), pp. 1–11.

INFLUENCE OF TRAM INDUCED VIBRATION ON UNDERGROUND GARAGE STRUCTURE

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Abstract

Light rail traffic presents significant advantages in a heavy populated urban environment in terms of quality of service, volume of passengers, delays etc. Vibrations caused by tramway vehicles passing through highly populated urban areas present a great impact on the quality of life of citizens. Beside the life quality, severe vibrations can also influence structural integrity of surrounding buildings and other infrastructure. In a crowded urban environment, underground structures such as underpasses, garages or water supply and sewage facilities are often located directly under the rail track structures. In such scenarios, special attention has to be pointed to track design in terms of vibration attenuation, as well as underground structure design in respect to additional dynamical loads presented by the overhead rail traffic. The paper presents a case-study of vibration measurement and analysis on an underground garage constructed under a tramway line on Kvaternik square in Zagreb. Rail discontinuities at turnouts and crossings present an impact point between tram vehicles and the track and therefore induce severe vibrations. To analyse tram induced vibrations and its effects on an underground garage structure, measurements on several locations have been conducted, including square surface and underground garage slab structures. Measurements have been conducted immediately after the construction of the garage and tram track on top of it, and repeated after 6 years of exploitation, using same principles and under same traffic load. This way the influence of exploitation wear and aging could be evaluated and analysed. Additionally, vibrations in the working environment of underground garage staff have also been considered. Such practice of periodical vibration measurements serves as an early warning system for any unexpected structural defects and provides a tool for future decision making and maintenance works scheduling.

Keywords: floating slab track, tram, vibrations, underground garage, exploitation

1 Introduction

Rail vehicles operation inevitably induces and emits vibrations to the environment. Main source of vibrations of the railway track structures originates from wheel – rail contact surface. In the urban environment the vibrations are transmitted through the ground to surrounding structures that are often found in close vicinity of railway tracks and as such can have a great influence on people living or working in the close vicinity and on structures themselves. To reduce the emission of railway noise and vibrations, two approaches are commonly used; applying mitigation measures to railway vehicles, or applying them to the railway track structure [1]. Taking railway vehicles into consideration, beside the usual geometry and wheel running surface maintenance, great attention has to be appointed to the vehicle suspension and type of wheels installed. For low speed tram and light rail systems found in urban

environment, the use of two-component wheels with elastic components is considered good practice. If railway track is considered, in addition to regular maintenance of running surface, the use of elastic fastening systems and continuously supported resilient tram tracks is recommended [2]. In order to regulate the level of vibrations people are exposed to in their living and working environment, methods of analysis and evaluation have been developed [3]. In the USA, the vibration exposure is covered by the FTA – Transit noise and vibration impact assessment [4], while in the EU, the directive related to vibrations in the working environment is 2002/44/EC [5].
The research consists of vibration measurements on tramway network of Zagreb Municipality Transit System (ZET). ZET operates around 180 tram vehicles along 116 km of narrow gauge tram track (1000 mm), forming the core of Zagreb public transportation system. Actual location of measurements is on a section of a tram track that passes over an underground garage at Kvaternik Square. The square serves as a turning and routing point for six tram lines, since three tramway routes intersect at the square. The tram track layout at the square includes turnouts and crossings, necessary for vehicle manoeuvring. Rail discontinuities at turnouts and crossings present an impact point between tram vehicles and the track and therefore induce severe vibrations. Goal of the research was to measure and evaluate the type and intensity of vibrations that are transmitted from the tram track to the underground garage underneath it. To analyse tram induced vibrations and its effects on an underground garage structure, measurements on several locations have been conducted, including square surface, underground garage slabs.
First vibration measurement and analysis has been carried out in 2008, immediately after the square reconstruction which included construction of the underground garage as well as the tram track on top of it, Fig. 1. The measurements have been repeated in the same way, 6 years into exploitation, in 2014 in order to record and analyse the difference of vibration propagation and intensity.

2 Description of test location

Kvaternik square has been reconstructed in 2006-2008. The project included construction of an underground 3 story garage underneath the surface of the square. Underground garage is a reinforced concrete structure resting on piled foundations. The project also included reconstruction of tram tracks on the entire square. Most of the tram tracks had to be completely rebuilt on top of the newly erected underground garage, Fig. 1. During the construction phase, the tram lines had to be diverted to different routes in order to clear the traffic of the construction site.

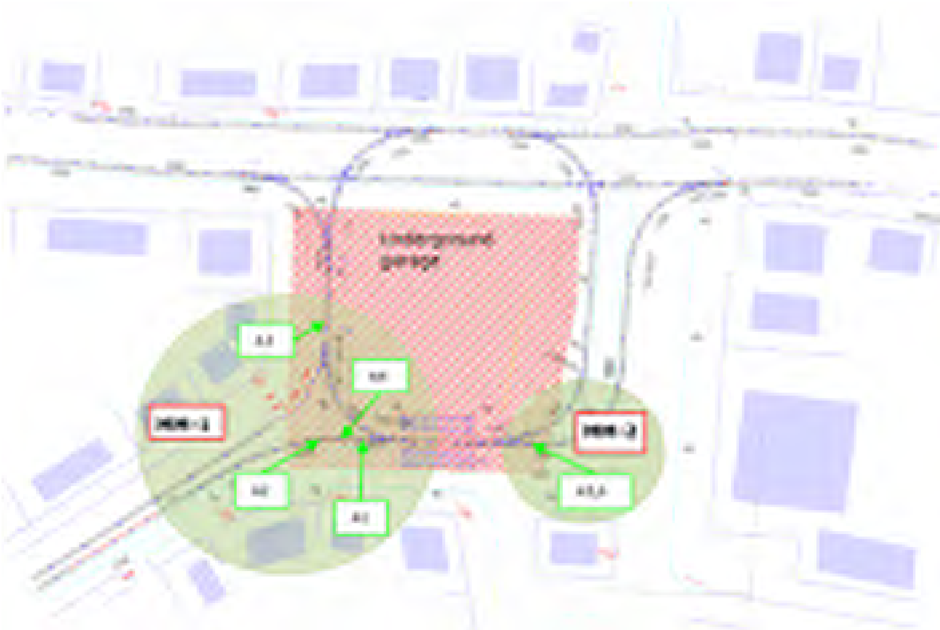

Figure 1 Kvaternik square layout with measuring locations

Since the new tram track rested directly on top of the garage upper concrete deck (at a ground level), measures for vibration attenuation had to be considered. For this purpose, the standard system of tram track in Zagreb (concrete slab, discrete rail supports and double elastic fastening system – DEPP) has been additionally fitted with continuous elastic supports underneath the rail and an asphalt layer beneath the concrete slab, Fig. 2.

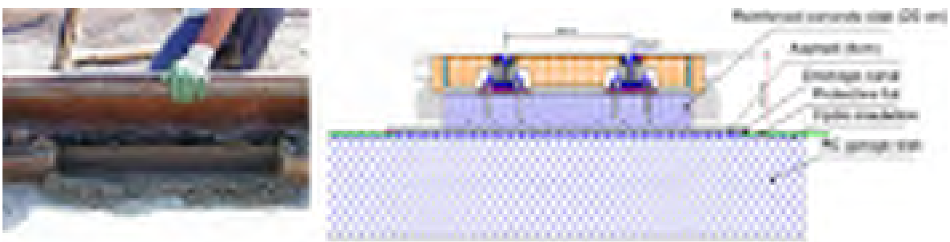

Figure 2 Cross section of floating tram track structure on top the garage slab of Kvaternik square with continuous under rail support (left) and asphalt levelling layer (right)

Continuous under rail support has been proven to reduce vibrations and noise in similar conditions [6]. The entire slab track structure rested on the poured asphalt layer, making it a floating slab track. Expected vibration attenuation of such track structures is between 10 – 25 dB, in 16 – 250 Hz frequency range [3]. Other characteristics of the system are the Ri60 grooved rails and paving of the track with reinforced concrete blocks.

3 Measurement and analysis procedure

Measuring points layout has been noted in Fig. 2. Two Measuring zones have been established – MM1 and MM2. Both zones are in close vicinity of the turnouts. Measurements have been carried out using 6 accelerometers (A1 – A6) fitted to the ground level (near the railway track), to the ceiling of the -1 garage level (surface RC slab) and on the floor of -1 garage level (-1 floor RC slab), visible on the garage cross section in Fig. 3.

Figure 3 Kvaternik square garage cross-section with measuring locations

All the vibrations have been measured in the vertical axis, perpendicular to the square plain. Since the location has quite heavy tram traffic, all of tram types running at ZET network have been recorded (T4, KT4, GT6, TMK2100, TMK2200, TMK2300). In the following analysis the focus is on two most frequent tram types recorded in 2008 and 2014, T4 and TMK2100 in order to make valuable comparison.

3.1 Data collection and presentation

Measurements have been carried out using Brüel & Kjær multichannel PULSE analyser. Vibrations of accelerometers A1 – A4 at MM1, as well as A5 – A6 at MM2 have been recorded simultaneously in order to evaluate the difference between surface vibrations at the source with the vibrations at different points in the garage structure. In 2008 a total of 32 tram passes have been recorded at MM1 and 18 at MM2. In 2014, 15 tram passes have been recorded at MM1 and 9 at MM2.

3.2 Data analysis

Time related accelerations have been recorded and connected to the passing vehicle. Each record has then been integrated in order to get speed of vibrations [mm/s] and spectral analysis has been performed in third octave bands in the range of 0 – 100 Hz.

3.2.1 Vibration propagation through the structure

MM1 zone has been established in order to record the influence of vibrations on different parts of the underground garage structure (see Fig. 1 and Fig. 3). It has been noted that the vibrations of the -1 floor (A2) have the higher value of measured vibrations that the upper slab

(A3, A4) of the garage. This was in conflict with the presumption that the vibrations would decrease the further we go from the source. However, due to the smaller construction height of the -1 slab, the vibrations have a greater influence on it than on the surface slab (Fig. 4).

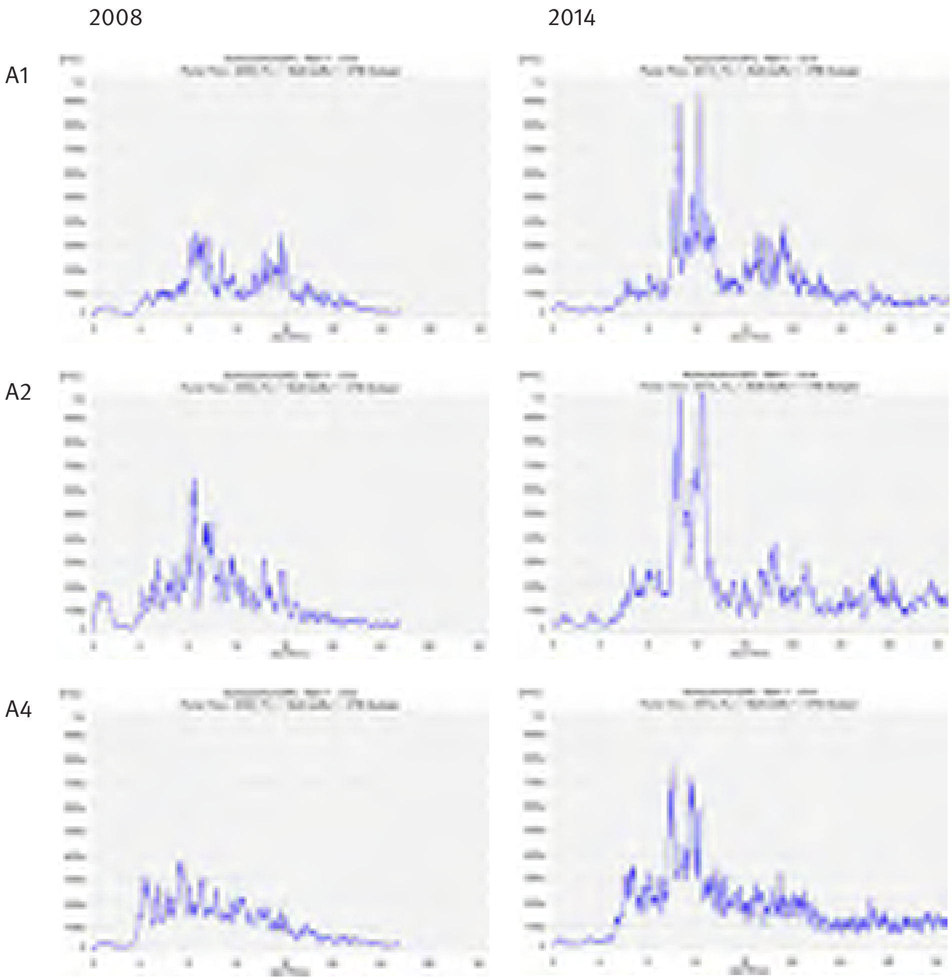

Figure 4 Speed of vibrations at MM1 (A1, A2, A4) for tram T4 in '08 and '14

It can be seen that the vibrations have increased in intensity over the 6 years of exploitation. As presented in one of representative measurements (pass by of T4 tram at MM1 (direction 1), increase in vibration speed can be observed (Fig. 4).
Increase in vibration intensity can be explained by the rail and turnout wear through exploitation, which results in surface irregularities and larger rail discontinuities at turnouts and crossings. This has been confirmed by a visual inspection and measurement of rail cross section. The intensity of vibrations has not increased to the point that it would cause any damage on the structure itself, but it could have an impact on the working environment of garage staff.

3.2.2 Vibrations in the working environment

Measurement zone MM2 has been positioned in the zone of a turnout that sits on top of a control room of underground garage staff. Accelerometers A5 and A6 have been fitted inside one of the control rooms in order to measure vibrations in working environment. Vibration recording of a TMK 2100 passing by MM2 (A5) has been presented in Fig. 5 for both 2008 and 2014.

2008 2014

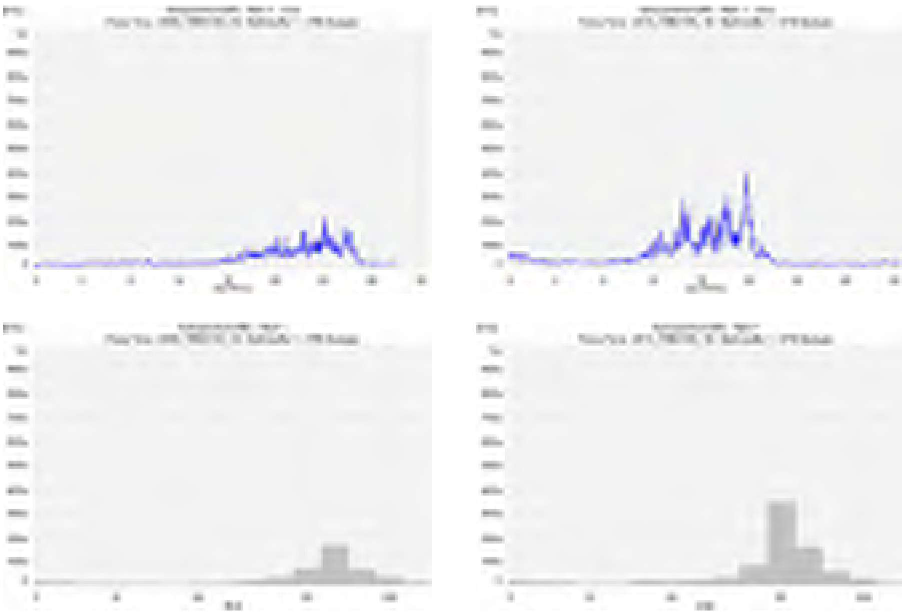

Figure 5 Measurements at MM2 (A5)

According to ISO 2631-2:1985, the measured value of vibrations in 2014 of 0.35 mm/s at 50Hz has passed the limit of 0.2 mm/s set by the standard. This is however just an indication, because 8h long measurements should be carried out and calculated as the highest (rms) value, or the highest vibration dose value (VDV) of the frequency-weighted accelerations, determined on three orthogonal axes, according to Directive 2002/44/EC.

4 Conclusion

Vibrations caused by railway vehicles are often a problem in an urban environment. At smaller frequencies they can produce squealing noise, and at larger frequencies, vibrations can disturb and harm humans in their living and working environment. Moreover, severe vibrations can threaten the structural integrity of the objects surrounding the railway line.
In Zagreb, ZET Municipality transit system is maintaining 116 km of tram track, mostly through highly populated urban area. An important design and maintenance task is to provide effective solutions for the tram track and tram vehicles in order to keep the low level of vibrations. In certain specific cases, like the one described in the paper, the tram track is located directly on top of the underground garage top slab. In order to achieve vibration attenuation properties of the track, a floating slab track has been designed and constructed under the supervision of the University of Zagreb Faculty of Civil Engineering. Such a demanding structure had yet to be evaluated in terms of vibration attenuation. Measurements have been conducted at the beginning of exploitation period in 2008 and in 2014, 6 years into exploitation.
Measurements and data analysis have indicated the vibrations have slightly increased over the exploitation period. The more distinguished peak values, and visual inspection of rail, turnout and crossing wear, indicate that the increase in vibrations is due to irregularities

and discontinuities of the rail running surface, especially on the tracks under heavier usage. Therefore, great attention should be appointed to the maintenance of running surface as well as turnout and crossing geometry. The overall vibration level has still remained relatively low and is not causing any structural damage.
Analysis of vibrations at MM2, in the working area of underground garage staff, an increase in vibrations intensity has also been noticed. To fully evaluate the level of vibration that workers are exposed to, extensive measurement and analysis should be conducted according to 2002/44/EC.
The goal of achieving greater attenuation of vibrations than on a standard tram track system installed in Zagreb has been reached by constructing a floating slab track with continuously supported rail and the asphalt pad between the track slab and underground garage slab. The practice of periodical vibration measurements serves as an early warning system for any unexpected structural defects and provides a tool for future decision making and maintenance works scheduling.

References

[1] Lakusic, S., Bogut, M. and Tkalcevic Lakusic, V.: Noise and vibrations at tram track intersection, Proceedings of 9th Congress of the French Acoustical Society ACOUSTICS ‘08, Paris, France, 2008.

[2] Thompson, D.: Railway Noise and Vibration, Elsevier, UK, 2009.

[3] Lakusic, S., Ahac, M.: Vibracije od željezničkog prometa, Gospodarenje prometnom infrastrukturom, 373-418, 2009.

[4] FTA, Transit noise and vibration impact assessment, Office of Planning and Environment, Federal Transit Administration. Report number: FTA-VA-90-1003-06, 2006.

[5] Directive 2002/44/EC of the European Parliament and of the council of 25 June 2002 on the minimum health and safety requirements regarding the exposure of workers to the risks arising from physical agents (vibration), 2002.

[6] Haladin, I., Lakusic, S., Bogut, M.: Analysis of tram traffic vibrations in respect to tram track structure and exploitation period, Proceedings of The 20th International Congress on Sound and Vibration, Bangkok, Thailand, 7–11 July 2013.

7 TRANSPORT GEOTECHNICS

STABILISATION OF FORMER TRUNK ROAD EMBANKMENT USING COMBINED STRUCTURAL AND ECO-ENGINEERING STRATEGIES

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Abstract

Mass wasting events and failures on the earthworks adjacent to trunk roads are becoming more frequent as a result of climate change and increased volume of transport. This paper describes the geotechnical aspects of stabilisation of a 750 m long and up to 55 m high coastal slope in Scotland bisected by a former trunk road with a history if instability and subsidence. Ground investigations, monitoring, specimen and detailed design considerations are discussed in the light of geo-environmental, financial, temporal, sustainability and risk considerations. The geotechnical design had to consider the local and global stability of the slope as well as the effect of potential climate change, land use and traffic volume changes. Additionally, the stabilisation options for the earthworks supporting the road were constrained by the requirements of the local authority and statutory undertakers. The detailed design for landslide and erosion prevention comprised extensive drainage measures and soil nailing designed to be installed within the existing slope surface, utilising an innovative head assembly, recessed within the slope and covered with pre-seeded biodegradable 'grow bags' making them 'invisible', providing a green slope finish, and minimising the visual impact of the works on the natural surrounding. The sustainable use of vegetation for slope stabilisation is highlighted through the use of eco- and ground bio-engineering strategies in the design and construction of stabilisation measures against shallow slope instability and erosion.

Keywords: sustainability, eco-engineering, soil nailing, slope stability, asset management

1 Introduction

A trunk road is a major road, usually connecting two or more places, which is the recommended route for long-distance and freight traffic. In the UK, trunk roads were first defined in the Trunk Roads Act 1936 when the Minister of Transport took direct control of 30 major roads (7,200 km, including bridges). Additional roads were 'trunked' either following the Trunk Roads Act (1946) or after being constructed as such. Trunk roads can be 'de-trunked', e.g. with construction of a motorway following a similar route, and then they become ordinary 'A' roads. The trunk roads in Scotland (currently 3,405 km or 6% of the total road network, carrying 37% of all traffic and 63% of heavy goods vehicles), have been managed separately since 1998. The trunk road and motorway network in Scotland has a gross asset value of approximately £18 billion. It is Scottish Ministers' single biggest asset.
Large number of embankments and cuttings not only on the trunk road network in UK are exhibiting signs of distress because of their age and historical lack of investment in maintenance and repair. The transportation and local authorities who own the assets are faced with decisions on balancing limited funding for maintenance and repair with investment for keeping up with the raising environmental standards and climate changes that threaten the assets'

stability [1,2]. Additionally, the lack of resources and understanding of the wider aspects of road maintenance and repair usually result in poor asset management and increase in the risk of failure of the slopes associated with a road [3, 4].
Slope instability events (landslide, translational slips, erosion) are natural events and can occur above or adjacent to sections of trunk roads. Major instability events such as landslides are most likely to occur during and immediately after periods of very heavy rainfall, especially if the heavy rain follows an extended rainy period. In Scotland, landslides are most prevalent in the periods July to August and October to January [5]. It is of critical importance that good quality information on the asset condition and their behaviour especially immediately before and after an instability event is needed to maintain their sustainability.
Traditional slope instability prevention and mitigation measures involve the use of structural elements and materials such as concrete and steel to provide safety against failure of the slope [3]. Recent trends in civil engineering also consider the use of live or living material for slope stabilisation such as vegetation. This approach, termed eco-engineering or ground bio-engineering [6] is based on a sustainable geotechnical design where the vegetation, apart from fulfilling an engineering function, contributes towards positive environmental and social impact at a relatively low cost.
In this case study, the stability of a former trunk road embankment in Scotland is investigated from the aspect of provision of sustainable geotechnical solution and the considerations informing the detailed design. Environmental and socio-economic aspects of the geotechnical design are outlined and the findings and obstacles associated with the design process are discussed.

2 Case study background and methodology

2.1 Site description and Background

The Bervie Braes comprises a coastal slope which lies immediately to the west of the old harbour in Stonehaven, Aberdeenshire. The Braes extend for approximately 850m and reach a maximum height of 55m. The former trunk road connecting Stonehaven to Dunnottar Castle and A94/A957 road runs sidelong and generally northwest-southeast across the slope, rising from the northwest end towards the at the far south-eastern end where it turns inland (Fig.1). Around 60 residential properties lie towards the toe of the slope with the residents using the vegetated road embankment for pedestrian access to the road and the nearby tourist attractions. The slope profile above the road is made up of a series of hollows, mounds, and bulges which represent washouts from ephemeral springs, failed material and areas of ongoing creep, respectively. The overlapping of old apparently inactive and active areas of slope instability provides the slope with a hummocky appearance.
The geological setting comprises alluvial raised beach deposits of gravel and sands overlying Glacial Till and, locally sands and gravels, of glacial origin. As a result of being on the periphery of a considerable fault (Highland Boundary Fault) the rock strata underlying Bervie Braes (Devonian conglomerates and sandstones) are deformed and reported as being highly inclined over the vast proportion of the Braes and vertical at the south end of the site. A number of ephemeral springs with a distinguishable seepage line exist on the embankment.
The Braes have a history of slope instability which is detailed on Fig 1. After the event of 2009, the local authority who owns the road commissioned a forensic study to determine the relationship between the local groundwater regime, including perched water levels, the locations of weak, sensitive soils and the movement and progressive failure of the road [7]. Based on this study, the specimen and detailed design for a stabilisation solution commenced in 2010.

2.2 Geotechnical design rationale

Structural stability aspect: The geotechnical design had to be based on provision of stability for the slope in both short- and long-term. Additionally, the slope stabilisation solution had to cater for both shallow and deep seated slope instability due to the complex geological setting and the fact that both of them have been observed as failure mechanisms on the slope in the past.
To inform the geotechnical design, a range of ground investigations (GI) were carried out and compared to the results of historic ground investigations on the site. The GI included non-intrusive and intrusive techniques described in detail elsewhere [7]. From these, the ground model including the characteristic values of the soil strength parameters was produced for a number of characteristic cross-sections of the slope. The GI also included the installation and monitoring of vibrating wire piezometers, standpipes and inclinometers. With these, the spatio-temporal characteristics of groundwater levels and movements within the soil mass could be observed and analysed. Potential failure modes were postulated [7] and a number of potential solutions (soil nailing, reinforced earth, retaining wall, drainage and combinations) investigated against the potential construction costs.
Environmental aspects: The environmental aspects informing the geotechnical design included consideration of the effects of the water and vegetation. Groundwater, on the other hand, was identified as the major cause of potential deep seated slope instability and had to be managed together with the surface water that would cause erosion or shallow slope instability. Hydrological and hydrogeological studies were carried out in order to ascertain if the amount of surface and ground water which is protected in the area [8] that would potentially have to be drained from the slope and discharged either into the existing surface water network or directly into the sea. This was followed by a video and geophysical survey of the existing drainage along and adjacent to the road which revealed disjointed pipes, blockages and a number of level and size irregularities which did not feature in the records of the local authority or the utilities company.

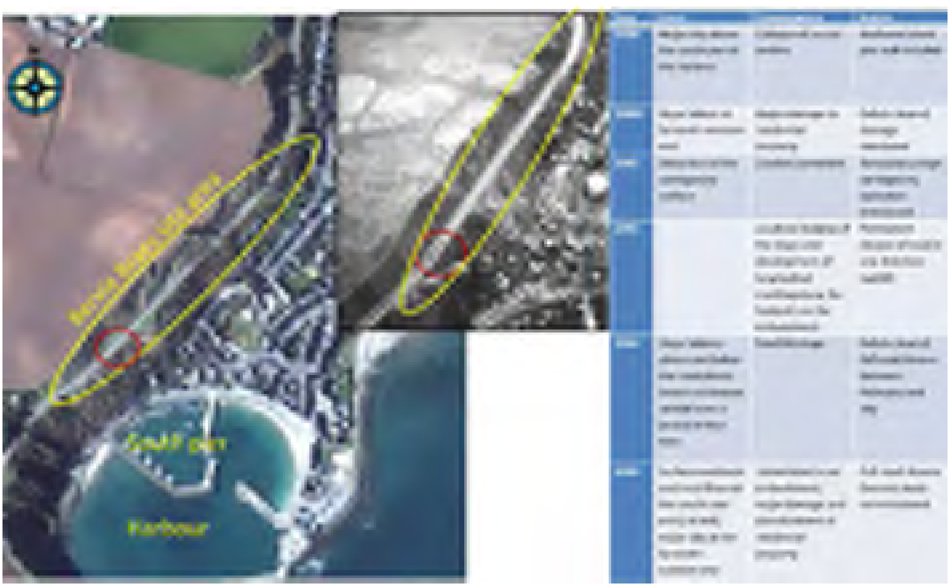

Figure 1 Satellite and historical aerial view of Bervie Braes (source Google Maps), together with an overview of historical slope instability events

The road embankment is part of the Stonehaven Conservation Area and major changes to the existing environment including the aesthetics are not permitted. To establish the baseline environmental conditions, ecological and dendrological surveys were undertaken. The ecological survey did not find evidence of protected species on the slope but noted that the trees and shrubs present on the slope host bird nests during the nesting period and construction should avoid this period. It was also noted that large parts of the slope are covered by seasonal vegetation which often leaves the ground exposed during the winter. The dendrological survey revealed relatively low variety of trees of low value on the slope which are liable to overturning.
Socio-economic aspects: The local authority who owns the road since it has been de-trunked consulted with Stonehaven Community Council who voted for the most acceptable stabilisation solution. This was important because the residents are one of the major groups that use the Braes for recreational purposes or access. Access was important for the residents as it brings tourists to Stonehaven while visiting the War Memorial (0.5 km south of the Braes) and Dunnotar Castle (3 km south) but also because this road is one of only 3 providing access to the town and the other two also are also threatened by slope stability problems. It should be noted that approximately 500 people live in the properties at the toe of the slope and have been at immediate risk of slope instability which motivated them to participate in the consultation. The majority of residents voted for the soil nailing solution for both the upper and lower slope, preferring full re-opening of the road after stabilisation.
After the events of 2009, the Scottish Government provided a grant which when combined with the funds available to the local authority could not provide stabilisation of the whole slope as voted for by the residents. A risk analysis resulted in a strategic decision to stabilise only the most critical sections of the road embankment leaving the slope above the road in its present condition [9].

3 Results – detailed geotechnical design

The detailed design was guided by the stability principles placed within a sustainability framework. Deep seated instability required stabilisation and anchoring into the more competent, deeper soil horizons while the short-seated instability required retention and protection of the ground surface. These objectives were achieved by designing soil nails toeing in the Glacial Till with concrete head assembly that would retain the more erodible sands near the surface (Fig.2). This structural solution also ensured the long-term stability of the slope with a design life of 120 years.
In the area that exhibited most distress and failure in the past (Fig 1 circled, inset), the carriageway was designed to be excavated together with the road base and subgrade down to beyond the failure surface and replaced with locally sourced suitably compacted material over which the new road base and wearing course will be constructed. With this, the design ensured that the risk of settlements due to internal erosion and washout of the subgrade will be significantly decreased.
The short-term stability of the soil around the soil nails which is exposed to erosion from surface water was ensured by the innovative design of the head assembly which is recessed into the slope and covered with bio-degradable bags containing soil and selected grass seeds. Additionally, the slope was hydroseeded after the completion of the soil nailing works with a selected mixture of seeds of native grass and herb species. This design, based on evidence and experience [6, 10] envisaged that the grass roots will grow quickly, reinforce the top layer of the soil while using water with their roots and thus provide protection against shallow seated instability.
Water as the major factor that potentially could cause slope failure was controlled by installation of raking drains near the toe of the slope whose function was to drain excess groundwater and discharge it, together with the surface water from the slope, into a contour drain channel

running along the toe of the slope and discharging into a piped drain that runs along the existing surface water drain and discharges in the open sea at levels that would ensure self-cleansing but also protection from flooding. New road drainage system was designed to capture the surface water from the slope above the road and thus limit the amount of surface water that could potentially reach and erode the road embankment. The drainage systems were designed with sustainability and resilience in mind with an increase in peak water quantities due to the effects of climate change taken into account.

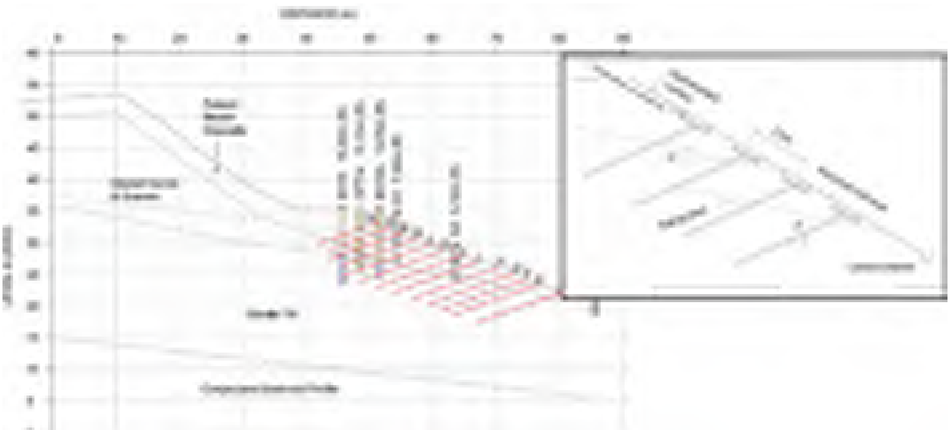

Figure 2 Detailed design of slope stabilisation measures: soil nailing with 'green' surface finish due to recessed soil nail head assembly. Drainage measures such as raking drains and contour channel (inset) also part of the detailed design

Another aspect of sustainability in the design was the requirement to cut the existing trees which were at risk of infection or failure and replace them with native species which will in long-term perform an engineering function such as soil reinforcement and water extraction form the soil with evapotranspiration. Additionally, the design envisaged planting of new 120 trees of native species to increase the biodiversity, provide wild bird habitat and increase the vegetation cover over the areas which could be exposed in the winter.
The detailed design considered the socio-economic constraints and specified phased construction in order to ensure partial access to the Braes for the residents and tourists with the choice of construction method left to the contractor in order to finish the construction within the set time period which, in turn, coincided with the non-nesting period for the birds. The design and specification were formulated using an innovative approach where the contractor was stimulated to achieve savings during the construction which are then used to procure more stabilisation works [9].

4 Discussion

The case study showed that the geotechnical design could not be disassociated from the wider environmental and socio-economic milieu. Although the close connection between geotechnics and sustainability exited in the past, only few studies have been published on sustainable design showcasing good (or bad) practice and increasing the profile of the profession [10]. It is hoped that this study will contribute towards the motivation of the geotechnical engineers to publish on the subject and actively work and collaborate in increasing the impact of sustainable geotechnical design.
The geotechnical design for this study was complex and time consuming. The complexity was demonstrated through the subtle interplay between structural stability issues and su-

stainability issues. While the structural issues could be resolved by numerical modelling and analysis, they required a number of parameters as an input which originated outside the strictly geotechnical domain. Parameters such as groundwater levels, precipitation patterns and quantities, creep movements required longer-term monitoring in order to reveal patterns and allow selection of critical values as an input into the geotechnical model or analysis. During the time these parameters were monitored, the risk of failure of the embankment and potential damage to life and property had to be managed by avoiding access to the slope and remote monitoring, almost real-time analysis of the remote monitoring data and issuing warnings to the residents and tourists. This process required resources and planning in addition to the investment for the equipment. However, the safety of the public and employees was very important for the local authority and the designer and the investment into the monitoring and warning system was negligible when compared to the cost of stabilisation works. Such system may be used on a temporary basis [4, 5] where risks can be calculated and a procedure exists for management of different levels of risk.

The approach of combining non-intrusive and intrusive GI was beneficial for the design as it provided good coverage of the site and correlation between the mechanical and physical properties of the soils on site. Proper planning of the GI within the budgetary restraints was critical as very little relevant historical data existed despite the history of failures on the site and the number of investigations carried out in the past. A shortcoming of this approach was the data resolution: a stonemasonry structure supporting the road in the most distressed section was not detected by the GI and caused problems and minor delay during construction. Similarly, geophysical survey was not able to clearly show a number of services running very closely together which, again, caused problems during construction. This issue could be efficiently resolved by combining the geophysics with video surveys where possible (as it was done with the drainage runs in this case study) but caution should be exercised when interpreting them together.

An interesting point was that vegetation was found to have a stabilising effect on the slope while back-analysing the existing conditions [7]. This encouraged the eco-engineering considerations in the geotechnical design and designing with vegetation which will perform some engineering function in the short- or long-term [6]. This approach contributed towards more lean and resilient design and also improved the aesthetics and ecological diversity in the conservation area. While recommending this approach, it should be noted that it should be followed by regular inspections and monitoring in order to ensure both engineering and environmental functions of the vegetation are performed.

The socio-economic constraints resulted in risk based design and an innovative use of NEC3 for procurement of stabilisation works with a limited budget [9]. The design and the coverage of the stabilisation measures had to be curtailed to within the budget limits which meant the areas with the highest risk of failure on the embankment supporting the road were stabilised first. This approach allowed good coverage of stabilisation measures on the embankment but left the upper slope in its existing state and several areas on the embankment without any stabilisation measures which still pose a risk of failure.

While the financial constraints dictated the magnitude of the works [4], it was the public who actually selected this remedial option which may not have been the most financially viable. In similar cases in the future, it may be interesting to question and record the motivation and concerns of the public when making such a decision as well as the way the options are presented to them. In the future, it may be more appropriate to assess and manage the risks using risk registers with quantified risks of failure and identification of criticality in order to provide a more solid basis for prioritisation and investment.

Lastly, the availability of critical information for the design was of great importance in this study. Although a number of studies with different foci were carried out on the site in the past 20 years there were no systematical inspection, maintenance or repair records available. The availability of such records and the easiness of access to them would have sped up the design process and decreased the number of assumptions made which, in turn would have resulted in lowering the factors of safety and getting more form the limited budget [3,4]. During the current study, new information was revealed and all the changes were recorded in a systematic way; however, the questions remain over the management and update of this data in the future within a modern asset management system [1,3].

References

[1] Meyer M.D., Amekudzi, A., O'Har, J.P.: Transportation Asset Management Systems and Climate Change, Transportation Research Record 2160, pp. 12-20, 2010.

[2] The Scottish Executive: Scottish Road Network Climate Change Study, eds. R.M. Galbraith, D.J. Price & L. Shackman. The Scottish Executive, Edinburgh, 2005.

[3] UKRLG Highway infrastructure asset management: UK Roads Liaison Group 2013.

[4] Fwa, T. & Farhan, J.: Optimal Multiasset Maintenance Budget Allocation in Highway Asset Management, J. Transp. Eng., 138(10), pp. 1179–1187, 2012.

[5] Transport Scotland: Scottish Road Network Landslides Study – Implementation, eds. M. Winter, F. Macgregor & L. Shackman. TS, Edinburgh, 2008.

[6] Norris, J.E., Stokes, A., Mickovski, S.B., Cammeraat, E., van Beek, R., Nicoll, B.C., Achim, A. (Eds.): Slope Stability and Erosion Control: Ecotechnological Solutions, Springer, pp. 290, 2008.

[7] Currie, C., Murdoch, W., Sallis, D., Smith, M.: Forensic investigation of a failed road at Bervie Braes, Stonehaven – models and mechanisms. In: Forensic Engineering – From Failure to Understanding, B. Neale (ed.), Thomas Telford , London , pp. 325-334. 2009.

[8] Natural Scotland: The river basin management plan for the Scotland river basin district 2009–2015, The Scottish Government, 2009.

[9] Mickovski, S.B., Black, J.D., & Smith, M.J.: Innovative use of ECC (NEC3) for procurement and management of infrastructure projects with limited funding: Bervie Braes case study In: Procs 29th Annual ARCOM Conference, 2-4 September 2013, Reading, UK, eds. S.D. Smith & D.D. Ahiaga-Dagbui, Association of Researchers in Construction Management, pp. 799-808, 2013.

[10] Mickovski, S.B.: Why is the future ready for environmental geotechnics?, Environmental Geotechnics [Accepted for publication], 2014.

POSSIBLE IMPACT OF EUROCODE 7 ON SLOPE DESIGN FOR ROADS AND RAILWAYS

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Abstract

Eurocode 7-1 covers the geotechnical design, among which is slope stability. As they are normally and routinely present in designing roads and railways, either as embankments or excavations, the necessity for actualization of Eurocode 7 through the prism of traffic infrastructures is understood. Most of the European countries have selected Design Approach 3 as appropriate for designing slopes according to Eurocode 7. Moreover, it was also found appropriate for Western Balkan countries, but in order to establish relation between the "old" and the "new" way of design, theoretical and numerical analyses were conducted. With them, direct relation between the global safety factor and prospective material partial factors has been adjusted, respecting both Eurocode 7 and former design tradition which has produced many stable and safe road/railway geotechnical structures. The results from research have shown that the material partial factor offered in Annex A of Eurocode 7-1 can not be directly applied if it is not absolutely clear in which circumstances i.e. load case, structure type, step of execution, reliability of parameters etc. it should be applied. This looks after enlarging the range of partial factors from the only single one as offered in Eurocode 7-1, since that keeping the single will push the roads and railways in the group of unsafe structures, unlike now. The methodology and findings are discussed further in the paper.

Keywords: Eurocode 7, slope stability, roads/railways, design tradition, relation

1 Introduction

Following the activities in the field of designing structures asks for accepting and application of Eurocodes (EC). In them, main news is limit state design, unlike hitherto working state design. There has been intensive work on the 10 parts of EC in the past decade, starting with their translation up to the most important part – preparation of National Annex. From that aspect, it seems that one of the most demanding was Eurocode 7 (EC7) which covers geotechnical works and it consists of two parts. The first one (EC 7-1) regulates the general rules for geotechnical design of foundations, retaining structures, embankments, their overall stability or hydraulic heave, while the second part (EC 7-2) is related to planning of terrain investigation, field and laboratory tests etc.

Slopes and their stability are part of EC7-1, and as slopes are commonly present in roads and railways, either through excavations or embankments, the necessity for actualization of EC7 through the prism of traffic infrastructure is understood. One of the specifics is that EC7 offers only single one material partial factor for determination of slope stability, suggesting its usage both in permanent and transient load cases, neglecting also the material and structure type. This differs drastically from the previous analyses and thus asks for careful calculation of slope stability in order to avoid endangering safety of traffic structures.

2 Slope design

2.1 Recent regulation

The factor of safety (FS) is defined as a ratio between the available shear strength and average mobilized shear strength needed to keep in equilibrium the hypotetic sliding body. It is calculated as a constant average relation of same value at all points of the sliding surface, although it does not have to be same for cohesion and tangent of angle of friction. The value of smallest allowable FS is determined in relation to the nature of an action, extent of investigations conducted on material properties etc. The standards [5] also specify that the FS increases with the lack of such conducted investigations. The determination of the FS may be regulated only locally, as it also depends on the environment (climate, type of ground, water, etc.)
Depending on the type of slope and material upon which it is performed, values shown in Table 1 are used as common design values for minimal FS, when highly reliable geomechanical parameters are available (those values correspond to actual standards [4] in the countries of former Yugoslavia still in the process of adopting Eurocodes):

Table 1 Values of applied partial factors

Type of structure	Shearing strength parameter	FS [min]
Embankment on hard or fairly compressible ground	Φ', c' or Φ'_B, $\Delta\Phi'$, p_N	1.4
Embankment on saturated ground with low bearing capacity	Φ', c' or Φ'_B, $\Delta\Phi'$, p_N; c_u	1.4; 1.4 ÷ 2.0
Excavation in saturated clays	Φ', c' or Φ'_B, $\Delta\Phi'$, p_N; c_u	1.4; 1.4 ÷ 2.0
Excavation in coarse grained materials	Φ' or Φ'_B, $\Delta\Phi'$, p_N	1.2

However, in a case of unreliable geomechanical parameters minimal values specified in Table 1 shall be increased by 0.25, while in a case of average reliability those shall be increased by 0.1. In addition, for highly reliable parameters it is recommended to apply the parameters Φ'_B, $\Delta\Phi'$, p_N and non-linear criterion of failure of hyperbolic type, instead of Φ', c' or Φ'_r [3]. Specified FS are applied in conditions of permanent and static loadings, so in the analysis of transient conditions of loading it is common to accept FS=1.30, while the FS for pseudo-static conditions is 1.10. However, the Standards do not differentiate between embankments/pits for roads and railways [4].

2.2 Eurocode 7

Eurocode 0 [1] suggests application of partial factors (PF) in analyses and designs; PF in EC7 factorize actions, material properties and resistances, and are imposed by the reliability of data, type of load, analyses, construction, cost effectiveness, maintenance and possibilities and effects of failure. Thus, depending on their position in equation, three design approaches (DA) are in use. The actual practice has shown the efficiency of "outdated" approaches through stability and functionality of structures, so the similarities between an "old" and new method allow adoption of relevant DA, while the approximation of dimensions of designed structures (with guaranteed safety) shall suggest the value of PF. The application of PF in analysis requires achieving of FS of 1.0. Comparative analyses have shown that the DA3 – among all approaches offered in Eurocode 7 – is the most appropriate for future analyses of slope stability [6], and it is also adopted in many European countries [9]. Within DA3, PF are applied on forces or effects and shearing resistance of the soil, thus only one calculation performed with values of input parameters is required. There are two types of PF for actions, depending on the fact if they originate from the structure or if they are of geotechnical origin, but in a case of slope stability and overall stability analyses, the effects over the ground (from any permanent load) are interpreted as geotechnical, thus using action PF=1,0. Different action PF i.e. PF=1,30, is used only for transient and unfavorable loadings [2].

3 Correlations

Until now, slopes were designed and analyzed according to the proved methods based on global factor of safety. The Bishop's method is one of the most popular methods for calculation of their stability in relation to the limit equilibrium. Besides many other advantages, this method is also acceptable from the aspect of EC7 [10]. Within this method, an iterative calculation is performed according to the following equation:

$$Fs = \frac{\sum_{i=1}^{n}\left[c' \cdot b + (W - u \cdot b) \cdot \tan\phi'\right] \cdot m_\alpha}{\sum_{i=1}^{n} W \cdot \sin\alpha} \tag{1}$$

where:

$$m_\alpha = \frac{1/\cos\alpha}{1 + \tan\alpha \cdot \tan\phi'/Fs} \tag{2}$$

Other methods vary mostly in regard to the manner of treatment of inter-laminated forces which, however, do not affect the selection of an approach and value of partial factors. Next step is determination of PF intended to reduce shearing resistance parameters (SRP); Annex A in EC7-1 suggests the value of 1.25. The condition for their determination is to provide same degree of stability prescribed so far, i.e., to assure same slope inclination, because the practice has proved their safety. If c=0 kPa and u=0 kPa, then Bishop's expression may be reduced to:

$$Fs = \frac{\tan\phi' \cdot m_\alpha}{\sin\alpha} \tag{3}$$

$$Fs = \frac{\tan\phi' \cdot \dfrac{1/\cos\alpha}{1 + \tan\alpha \cdot \tan\phi'/Fs}}{\sin\alpha} \tag{4}$$

$$Fs = \frac{\tan\phi'}{\sin\alpha \cdot \cos\alpha + \sin^2\alpha \cdot \tan\phi'/Fs} \tag{5}$$

Eqn (5) implies that:

$$\tan\phi' = Fs \cdot \tan\alpha \tag{6}$$

If eqn (5) is applied in conditions of limit state (when FS=1.0), then $\tan\Phi'$ should be replaced with design value of the angle of friction derived from the condition:

$$\tan\phi_d = \frac{\tan\phi'}{\gamma_\phi} \tag{7}$$

so the expression for FS would be as follows:

$$Fs = \frac{\tan\phi_d}{\sin\alpha \cdot \cos\alpha + \sin^2\alpha \cdot \tan\phi_d / Fs} \tag{8}$$

knowing that FS=1.0, this eqn (8) would imply that:

$$\tan\phi_d = \sin\alpha \cdot \cos\alpha + \sin^2\alpha \cdot \tan\phi_d \qquad (9)$$

which, after transposing tanφ' to the left side and rearranging trigonometry expressions would gain that:

$$\tan\phi_d = \tan\alpha \qquad (10)$$

$$\frac{\tan\phi'}{\gamma_\phi} = \tan\alpha \qquad (11)$$

$$\tan\phi' = \gamma_\phi \cdot \tan\alpha \qquad (12)$$

The equalization of the last two expressions with tanΦ' on the left side (eqns (6) and (12)) would gain:

$$Fs \cdot \tan\alpha = \gamma_\phi \cdot \tan\alpha \qquad (13)$$

and, finally:

$$\gamma_\phi = Fs \qquad (14)$$

According to fore mentioned it can be stated that the PF that shall reduce the tangent of friction angle [γ_Φ] in limit state conditions – while maintaining the same degree of safety previously provided with the global FS – is equal to the actual FS; the latter depends on the material, type of structure, nature of loading and design situation in which stability of the slope is being observed: permanent, transient or accidental condition.
Similar method was also used to prove that the same stands for cohesion (whether effective or undrained), which is also being verified [8].
The described approach for determination of partial factors allows this analysis to be used both in application of a Mohr-Coulomb law and non-linear failure envelope of hyperbolic type [3]. Non-linear interpretation of shearing resistance – due to realistic description of all types of soil within overall range of stresses – is particularly helpful during the optimization of slopes. Above all, this stands for soils with coarse grained materials, which apparently show critical stability of shallow surfaces to shear (and therefore require most of the engineering attention), but also for regular dimensioning and prevention of eventual failures at slopes with fine grained materials. Both instances derive from imperfections of a Mohr-Coulomb linear envelope in zone with low normal stresses, which are commonly competent for determination of SRP required for analysis of slope stability: it overestimates the strength of fine grained materials and underestimates the strength of coarse grained materials.
Related to this nonlinear failure envelope (Firuge 1), it was also found that it can be implied with partial factors [6, 7].
The described approach and findings allow a slope with previous global factor of safety of 1.50 to gain factor of 1.0 in the limit state. In both cases the specified values of factors will relate to the same sliding surface. So, several procedures have proved that in calculation of slope stability the value of partial factors by which SRP are reduced is equal both for the tangent of angle of friction and cohesion, and is also equal to the global factor of safety. However, besides on the proper parameters, its intensity also depends on actual conditions in which the slope stability is

examined: permanent, transient or accidental; they also may be distinguished in relation to the type of structure (dam, road, mine, etc.). Eurocode 7 Annexes do not offer such categorization of values of PF, so they are proposed as constant and independent from particular case, which poses one of its rare shortcomings. However, some possibilities are partially mentioned in Eurocodes 0 and 1. Those Eurocodes allow variations in PF based on the extent of consequences and reliability, but also on design situation, i.e., on load case, which is particularly appropriate to our recent practice! Proposed values maintain the actual and confirmed degree of safety, i.e., the same slope inclination, which is particularly important in a case of possible rehabilitation and upgrade, thus avoiding any future threats to the slope usability and allowing comfortable accommodation of engineers to the new design approaches according to EC7.

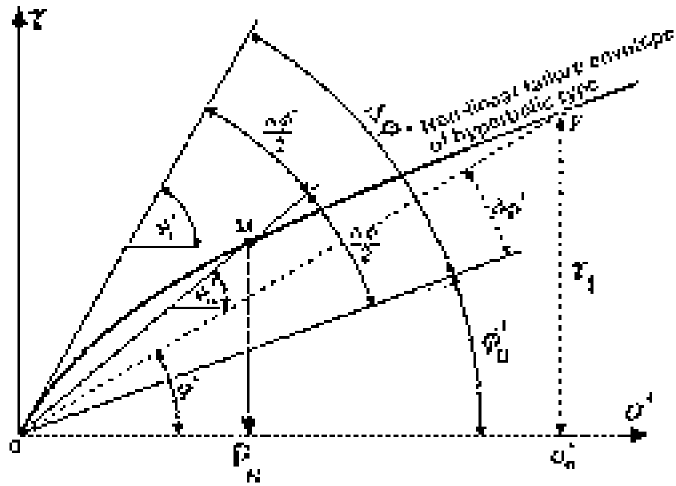

Figure 1 Non-linear failure envelope of hyperbolic type and its parameters.

4 Conclusions

In order to determine the relation between "old" and "new" design approach, some theoretical analyses were performed. The findings were confirmed with numerical calculations. This established direct relation between the actual factor of safety and future material partial factors, while respecting the Eurocodes and actual designing practice. Moreover, the results gained from investigations are appropriate for Mohr-Coulomb linear and non-linear failure envelope.
It was concluded that analyses of slope stability in our region (former Yugoslavia) require Design Approach 3, also adopted in many European countries. However, the value of PF for SRP is not constant as offered with Eurocode 7, but variable and dependable on the material, reliability of parameters, load cases etc. Thus, in order to keep designing safe slopes at roads and railways, following the analyses shown, it can be concluded that SRP in our region – when calculating the stability of excavation or embankment slopes with different types of materials – and provided that parameters have average reliability, during their application in DA3 should be reduced by:

- 1.50 for excavations in saturated clays and embankments (regardless to the base material) for permanent load conditions (1.40 for high reliability of SRP)
- 1,30 for excavations in coarse grained materials (1.20 for high reliability)
- 1.30 for analyses in transient load conditions
- 1.10 for pseudo-static analyses
- to maintain original values, in case of reverse analysis, because of reduction by 1.0.

Although it seems that these factors are higher than the primary offered in EC7-1, it has to be said that they are needed such in circumstances of initial application of EC7. Namely, in order to introduce EC7 to the new users, either as concept or from aspect of results, they have to have these values as only they keep the proved stability and slope. Their reduction should be expected, but only after several years of continual application of the proposed design approach and partial factors, followed by monitoring of such designed and built slopes and with respecting of the recommendations for quality and number of field and laboratory tests as stated in EC 7-2

As it is extensively described and commented in papers [6], [7] and [8], the specified approach and identical value of coefficients are rather favorable and recommended for analysis of slope stability in computer applications based on the finite element method, as well for modeling of materials using non-linear failure envelope of hyperbolic type.

Reference

[1] Evrokod 0, Osnove proračuna konstrukcija, Građevinski fakultet Univerziteta u Beogradu, Beograd, 79p, 2006

[2] Evrokod 7, Geotehnički proračun, Deo 1: Opšta pravila, Građevinski fakultet Univerziteta u Beogradu, Beograd, 163p, 2009

[3] Maksimović, M.: Mehanika tla, IV edition, AGM knjiga, Beograd, 643p., 2001

[4] JUS U.C4.200-1990. Projektovanje puteva: Osiguranje onjeg stroja saobraćajnica i ukupne stabilnosti tla i trupa puta. Geotehnika i fundiranje, Knjiga 6-2, Zbirka jugoslovenskih pravilnika i standarda za građevinske konstrukcije, Građevinski fakultet Univerziteta u Beogradu, Beograd, pp. 806-834, 1995

[5] JUS U.C5.020-1980. Projektovanje nasutih brana i hidrotehničkih nasipa. Tehnički uslovi. Geotehnika i fundiranje, Knjiga 6-2, Zbirka jugoslovenskih pravilnika i standarda za građevinske konstrukcije, Građevinski fakultet Univerziteta u Beogradu, Beograd, pp. 937-954, 1995

[6] Papić, J. Br., Dimitrievski, Lj., Prolović, V.: Odgovarajući pristup i parcijalni faktori za analizu stabilnosti kosina po Evrokodu 7, 4. Internacionalni naučno-stručni skup "Građevinarstvo – nauka i praksa", Žabljak, pp. 1333-1340, 2012a

[7] Papić, J., Dimitrievski, Lj., Prolović, V.: Value of partial factors for EC7 slope stability analysis: solved "mystery"?, 3rd International Conference on New Developments in Soil Mechanics and Geotechnical Engineering, Nicosia, pp. 193-198, 2012b

[8] Papić, J. Br., Dimitrievski, Lj., Prolović, V.: Verifikacija izabranog projektantskog pristupa i parcijalnih koeficijenata za analizu stabilnosti kosina po Evrokodu 7, 2nd International scientific meeting "State and trends of Civil Engineering", Tuzla, pp. 607-614, 2012c

[9] Schuppener, B.: "Eurocode 7: Geotechnical design – Part 1: General rules – its implementation in the European Member states", International seminar on Eurocode 7 and geotechnical aspects of Eurocode 8, MAG-ISSMGE, Struga, pp. 20-30, 2008

[10] Simpson, B., Driscoll R.: "EC 7 – a commentary", CRC, London, 1998

GEORISK – A RISK MODEL AND DECISION SUPPORT TOOL FOR RAIL AND ROAD SLOPE INFRASTRUCTURE

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Abstract

This paper presents a risk analysis and decision support tool developed for infrastructure assets on both rail and road earthwork networks. This three-stage risk management framework, called GEORISK, is specifically targeted at evaluating slope stability for cutting and embankment assets, focusing on slope stability problems, and is initially being developed for the Irish railway network.
The first step in the framework involves compilation of existing data available to infrastructure owners in a structured database of input parameters that define the controlling variables. This step also involves initial risk modelling to assess the probability of failure of the slope assets. The output of the first stage are probabilities of failure for each of the network's assets.
In the second stage, the probability of failure is subsequently refined using a condition degradation factor that allows non-standard evidence to be incorporated into the analysis. The probability of slope failure is then combined with a vulnerability analysis to determine the consequential impact of the failure. This allows various traffic and loading related factors to be considered.
In the third stage, a slope asset management plan is developed to include mitigation and remediation strategies. This includes a cost benefit analysis (CBA) tool that can be used in parallel with the slope management plan to inform decisions on where expenditure should be focused, offering value for money on annual maintenance budgets.
Overall, the GEORISK tool allows allows the key stakeholders and infrastructure managers to move from a system of reactive maintenance and towards targeted allocation of annual budgets for the highest risk assets.

Keywords: risk management, road and rail networks, geotechnical assets

1 Risk management for slope infrastructure – introduction

Landslide risk management as an interdisciplinary geoscientific topic has been extensively researched from the early 1970's. Recent advances in GIS and other software [1] have made it possible to ease the implementation of the risk management tools over large geographical areas and linear infrastructure networks, thus making them a highly usable tool for infrastructure managers and stakeholders.
Whilst a number of landslide risk assessment and management methods have been proposed, they all follow a similar structure on a macro scale. An example of a typical landslide risk management flow diagram is given in the Figure 1 [2]. Hazard assessments are a starting point for every risk assessment. These deal with characterization of landslide events and the determination of the probability of occurrence of given event. Various approaches can be used to calculate probabilities of failure. A wide range of hazard maps, including landslide inventory and landslide susceptibility maps, are also usually produced as an output, [3, 4].

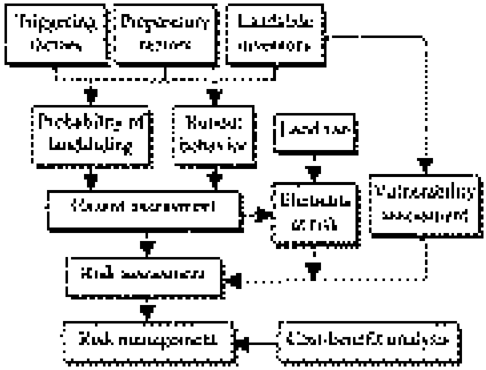

Figure 1 Landslide risk management flow diagram [2]

To complete the risk analysis, consequence analysis that includes vulnerability assessment and identification of elements at risk has to be combined with the hazard assessment, as risk is commonly defined as a product of hazard, vulnerability and elements at risk [5]. Vulnerability assessment defines the degree of loss or damage to a given element at risk affected by the hazard, and takes values on the scale between 0 and 1 [6].

Recent extensive reviews have focused on various levels of risk management like hazard assessment, risk assessment, and full risk management [2], [3], [7]. However, most of the landslide risk management procedures and methods produced have been developed to address the landslides on the natural slopes [8]. Even though the mechanisms of failure are largely the same for engineered slopes, certain aspects of the assessment of infrastructure networks require special consideration. Many major transport infrastructure owners use internally developed risk assessments and management tools, designed to address the specific needs and conditions present on the given network.

In this paper, a risk management model and decision support tool recently developed for cutting and embankment assets on the Irish rail network is presented. The main advance brought by this method is the switch from a visual risk assessment, frequently used by road and rail operators, into a highly sophisticated and less subjective approach with high-end geotechnical calculations taking advantage of data which is available to the infrastructure managers. The risk model is versatile enough to be applied to all rail and road infrastructure networks in general.

2 Risk model Background

The Irish Rail network was among the first rail networks to be constructed with the majority of the network dating back to the mid-1800s. As a result of this, a significant proportion of the network is comprised of aged cuttings and embankments. Like much of the rail infrastructure across Europe, the Irish network predates modern design standards and was built using basic construction techniques and readily mixed local materials. A significant proportion of the network, which has remained stable for over a hundred years, has slope angles far in excess of current design recommendations. Many of these steep slopes are also constructed at angles in excess of the material's natural constant volume friction angle. Their stability is provided by transient suctions which makes them particularly susceptible to rainfall induced shallow failures. In light of changing climate conditions the incidence of slope failures is likely to increase. Therefore to ensure optimum investment, it is imperative that infrastructure managers can quantify the risk represented by individual earthwork assets and rank them accordingly. This will allow for strategic investment to ensure optimum value for Irish Rail in terms of both cost and safety.

3 Stage One – Data requirements and initial risk model

3.1 Data requirements and failure modes

The first phase of the framework is predominately concerned with data collection and subsequent database population. This involves collating existing asset information into a manner which can interface seamlessly with the risk model and compile additional material from external sources. To facilitate this, it is first necessary to identify the parameters critical to earthwork instability for each failure mode considered. This includes not only the geometric and material parameters, but also includes the natural and anthropogenic triggering factors that may lead to instability. Once this has been accomplished the parameters are broken down into lists of known variables and variables which can be inferred based on known parameters. An example of a variable which is known is slope angle. Whereas, an example of a variable which can be estimated with a reasonable level of confidence based off of national soil maps and existing geotechnical knowledge, is the internal angle of friction of a soil. Each parameter will have an associated coefficient of variation which measures the parameter variability. When all variables have been identified, the database is expanded appropriately to account for new variables. The database is then populated using all available resources. In the case of Irish Rail, additional information was gathered from LiDAR scans, national soil maps, national borehole databases, site inspection reports and geological surveys. Naturally, if a parameter is inferred based off of known parameters it's coefficient of variation is increased to reflect its origin.

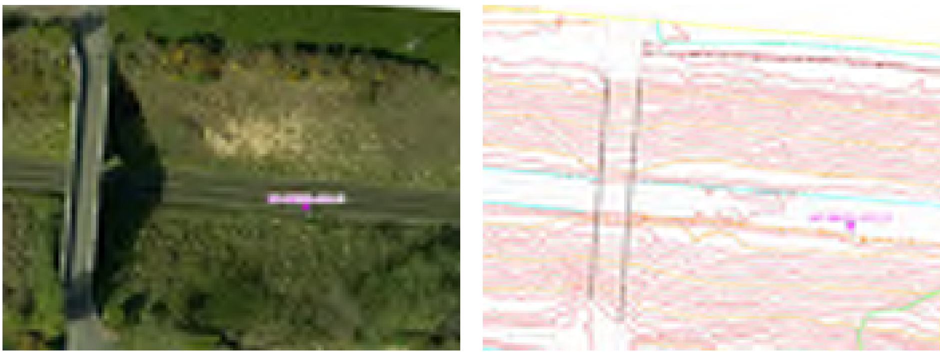

Figure 2 Example of possible various dataset layers gathered from surveys

Soil slopes and in particular rock slopes are susceptible to a wide array of different failure mechanisms. For the purpose of this project a desk study was carried out to determine the failure modes most likely to occur for Irish slopes constructed from glacial till and it was determined that shallow translational slips were most likely to occur due to the large volume of rainfall which occurs annually in Ireland.

However rotational and wedge failures were considered as there is increased consequence associated with these failure modes due to the larger volumes they displace. Shallow translational slides normally result from rainfall infiltration and are generally superficial in nature with the depth of the slide being significantly less than its length, while rotational failures generally occur at depth with an approximately circular slip path. Rotational failures usually arise due to excessive loading in areas with low internal angles of friction or due to some other form of changing boundary condition. They have a much larger volume than translational slides and are usually slow moving. However, in some weak cohesive clays there can be an extremely rapid run-out. Rock slopes on the other hand are more susceptible to wedge failures along pre-existing cracks or faults. These failures vary in magnitude and usually occur rapidly.

Figure 3 Observed slope failure modes

3.2 Probabilistic Model

Due to the heterogeneous composition of soil, huge variability can exist across relatively small sites [9]. In traditional deterministic analysis, factors of safety are calculated using either the mean parameter values or values slightly less than the mean. However this may not necessarily be conservative, for sites with huge variability this approach can actually overestimate the safety. Using a probabilistic based approach, two slopes which appear identical in terms of geometry and mean geotechnical parameters, could have vastly different probabilities of failure based on their natural variability. Probabilistic tools are extremely useful for extending the service life of existing infrastructure as they give a more accurate representation of stability, allowing designers to classify some assets which struggle to meet modern deterministic safety factors as safe and reliable. Furthermore from an economic point of view it is unfeasible to replace large sections of road and rail infrastructure. Therefore it is necessary to be able to classify the relative risk associated with each asset and fix critical infrastructure first.

In the GEORISK framework, the Hasofer Lind [10] first order reliability method (FORM) is used to calculate the probability of failure associated with each asset and its coupled limit state. The Hasofer-Lind approach is an invariant method for calculating the reliability index β, which can then be transformed into a probability of failure p_f. The first step using this methodology is to transform all variables into normalised random variables. This is accomplished by means of equation (1).

$$\overline{X_i} = \frac{x_i - E[x_i]}{\sigma[x_i]} \qquad (i = 1, 2, ..., n) \tag{1}$$

After normalising the variables the next step is to express the limit state in terms of the reduced normal random variables, as in eqn. (2)

$$g(\overline{X}) = g(\overline{x_1}, \overline{x_2}, ..., \overline{x_n}) \tag{2}$$

In this reduced variable space the limit state surface $g(\overline{X})$=0 describes the boundary between stable and unstable zones. The Hasofer Lind reliability index in then expressed as the minimum distance between the origin (the mean value of the reduced limit state) and the failure zone. The point on the limit state surface which is closest to the origin is known as the design point. The distance to this point can be described using the following equation (3) [11].

$$\beta_{HL} = \min_{\overline{X} \in \psi} \left\{ \overline{X}\,\overline{X}^T \right\}^{1/2} \tag{3}$$

where:

$\overline{X}$ vector representing the set of reduced random variables;
Ψ failure region defined by letting the performance function $g(\overline{X})$=0.

By constraining this equation the user is able to obtain the reliability of the limit state at the design point. Assuming normal random variables, a probability of failure can then be obtained using the following equation (4).

$$p_f = p\left[g(\bar{X}) < 0\right] = 1 - \phi(\beta) \tag{4}$$

where:

$\phi(\cdot)$ standard normal cumulative distribution function.

Randomly selected assets are then subjected to Monte Carlo simulation for verification.

4 Stage Two – Model refinement and Vulnerability assessment

4.1 Model refinement and Degradation factor

In phase one, the probability of failure is calculated from preparatory variables using inputs like slope geometry and soil types and strength parameters. However, calculation of the probability of failure using this data still does not take account of two important groups of data: the current slope condition and data related to landslide triggering events.
The actual slope response is controlled by variables which cannot be easily described. These variables include data that is usually recorded in a qualitatively manner, such as: type and condition of drainage, type and density of the vegetation, slope erosion and overall condition, etc. Usually they are collected through road and rail infrastructure operators' internal investigations and visual assessments. These factors thus need to be quantified prior to inclusion in the risk model. This is done by introducing the Degradation Factor, which assigns numerical weightings for each qualitative variable and adjusts the probability of failure obtained from stage one. "Hotspots". The past slope history is also accounted for at this stage, allowing previous failures or remediation works to be incorporated into the analysis through an adjustment to the raw P_f. This process also allows the model to be live, with subsequent failures or corrective actions being incorporated into the network wide risk model and the overall relative ranking recalculated accordingly.
Rainfall is by far the most important landslide trigger across Ireland, on the road, rail network or natural landscape. For that reason, rainfall values are considered directly within the database and are inputted in the risk model which will also account for seasonal variability in precipitation. Another triggering input which is included in P_f calculation refinement is surcharge loading.

4.2 Vulnerability assessment

In order to proceed from the hazard assessment done through a refined P_f calculation to risk analysis, a vulnerability assessment must be completed for the elements at risk on the asset network. This analysis includes the cuttings and embankments themselves as well as adjacent objects and structures on the line.
For that reason, information on line ratings, line speeds, the number of tracks, flow, passenger density, and other traffic related data are to be obtained and assigned. The information on adjacent objects' (stations, buildings, adjacent land use) and clearances are also necessary to evaluate the impact of possible landslides. The level of impact can be obtained through the inventory of historical failures and subsequent damage (an example of which is given in the Figure 4), as well as through scenario modelling.

Figure 4 The effects of slope failure of 31/12/2013 on Waterford rail station

Finally, a risk value for each asset can be calculated as a function of hazard and vulnerability assessment outcomes, and the ranked list of assets is compiled.

5 Stage Three – Decision support tool

After the risk values are assigned to each asset, a decision support tool will be developed. This will result in an array of possible answers and procedures for risk reduction. This process will involve some preliminary engineering to develop a slope asset management plan that incorporates generic remediation and mitigation strategies for slopes with different risk profiles. The slope management plan will be developed using an iterative approach that allows the user to test the impact of different maintenance strategies on the long-term risk rating of the assets in the network. Possible scenarios, such as modification of slope geometry, additional geotechnical investigation work for uncertainty reduction, installation of drainage, retaining structures, slope reinforcement, detailed finite element analysis or installation of slope monitoring equipment can be proposed. This will allow the slope management plan to cope with high risk assets that need urgent intervention using hard engineering solutions and also considering more strategic investment decisions that may change the national risk profile (e.g. installing simple drainage on entire set of asset classes).
A cost benefit analysis tool will be developed as an independent module that can be used in parallel with the slope management plan to inform decisions on where expenditure should be focused to maintain minimum safety standards and simultaneously offering value for money on annual maintenance budgets.

Figure 5 Methodology flowchart

References

[1] van Westen, C.J.: Geo-Information Tools for Landslide Risk Assessment: An Overview of Recent Developments, Landslides: Evaluation and Stabilization, Proceedings of the 9th International Symposium on Landslides. Rio de Janeiro, pp. 39–56, 2004.

[2] Dai, F., Lee, C. & Ngai Y.: Landslide risk assessment and management: an overview, Engineering Geology, 64, pp. 65-87, 2002

[3] Aleotti, P. & Chowdhury, R.: Landslide hazard assessment: summary review and new perspectives, Bulletin of Engineering Geology and the Environment, 58, pp. 21-44, 1999.

[4] Guzzetti, F. et al.: Landslide hazard evaluation: a review of current techniques and their application in a multi-scale study, Central Italy, Geomorphology, 31, pp. 181–216, 1999.

[5] Varnes D.: Landslide hazard zonation – a review of principles and practice, UNESCO, Paris, pp. 63, 1984

[6] IUGS Working Group on Landslides: Quantitative risk assessment for slopes and landslides— the state of the art, Landslide Risk Assessment eds. Cruden D. & Fell R., Balkema, Rotterdam, pp. 3-12, 1997

[7] Fell, R. & Hartford, D.: Landslide risk management, Landlside Risk Assessment eds. Cruden D. & Fell R., Balkema, Rotterdam, pp. 51-109, 1997

[8] SAFELAND Work Area Summary, www.safeland-fp7.eu/results/Pages/SafelandWorkAreaSummary.aspx, 22.03.2014.

[9] Phoon, K. & Kulhawy, F.: Characterization of geotechnical variability, Canadian Geotechnical Journal, 36(4), pp. 612-624, 1999.

[10] Hasofer, A. & Lind, N.: An exact and invariant first-order reliability format, Journal of Engineering Mechanics, 100(1), pp. 111-121, 1974.

[11] Low, B. & Tang, W.: eliability analysis using object-oriented constrained optimization, Structural Safety, 26(1), pp. 69-89, 2004.

SLOPE REMEDIATION METHODOLOGY ON THE ZAGREB-MACELJ HIGHWAY

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Abstract

This paper describes methodology used in slope maintenance in cut area on 18.5 km Krapina – Macelj section of Zagreb-Macelj motorway. Based on geotechnical investigations 15 cut locations were treated in main design and for another 15 locations detailed design is made. After motorway opening annual monitoring program was conducted. Program included measurement interpretation, visual observation of slopes and periodical reports. Report contained description of observed slope instabilities and defects and proposed one-time and periodical maintenance measures. The report conclusions initiated slopes remediation. This paper show 2 cut locations, Gornji Macelj 4 and Straža-jug. After defect detection detailed visual observation was done and crackmeters were installed. Based on displacement measurements and defect description back-analysis of slope stability was conducted. The result of analysis was remedial measures on slope protection system. Adaptation of slope stability protection within periodical maintenance allows on time detection of potential high risk situation, damage prevention and the use of optimal remediation measures.

1 Introduction

This paper describes design methodology and implementation of rock slope protection and stability measures on the Zagreb-Macelj motorway. Final slope protection is based on investigation works, local site experience, measurements and monitoring during construction and exploatation. The A2, Zagreb-Macelj, motorway is part of the Croatian motorway network and Pan-European Corridor Xa. It is part of Pyhrn motorway (Nürnberg–Graz–Maribor–Zagreb), which connects Croatian and European motorway networks. This paper describes the design solutions, slope excavation and exploatation phase for two locations: Straža Jug and Gornji Macelj 4, and also slope protection modifications based on the monitoring results.

2 Slope stabilization system

Straža Jug slope location is built of a limestone and a tuffaceous dolomite. These deposits are massive. Limestone is crystalline with dark gray-brown color and intersected with veins of calcite. Upper 4.0 – 5.0 m is an overlying clay layer with fragments of base rock material. Cut has four floors, each 10 m high with 4 m wide bench between them. Slope inclination of each floor is 2:1. On the lowest slope clogged perforated pipes were installed at 2.0 x 2.0 m grid. Slope was protected with shotcrete and the anchors. On the northern edge of the cut slope inclination mitigates from 2:1 to 1:1.5 and fits with the natural terrain outside the highway fence. Toe of the first floor slope finished on the embankment around the bridge abutment. Because of that the slope protection was freely hanging on that part of the slope.

Gornji Macelj 4 slope location is built of mostly a fine-grained sandstone covered with a weathering rind, 1.0-5.0 m thick. An overlying crust consists of clayey sand with fragments of base rock material. Crust thickness is about 3.0 – 4.0 m. Figure 1 presents the slope during excavation. There is a visible boundary between the weathering rind and the sandstone bedrock. During the exploatation, the cracks in concrete revealed locations with the thicker weathering rind. Upper crust had displacement on the contact with the bedrock.

Figure 1 Gornji Macelj 4 cut during construction.

Gornji Macelj 4 cut has one bench. The first, lower slope was excavated in the 2:1 slope and protected with shotcrete and the anchors. Southern part of the slope was mitigated and covered with stone concrete wall. Anchors were placed in two rows at 3.0 x 3.0 m grid. In the upper row 6.0 m long and 3.0 m long in lower row IBO self-drilling anchors were installed. Between anchors clogged perforated pipes were installed for drainage. At the peripheral areas of first slope floor MacMat with anchors was installed. The second, upper slope has 1:1 inclination and was protected with MacMat and 6.0 m long anchors. In the highest cross-section second floor has 4 anchor rows. Between the two slope floors there is 4.0 m wide bench with the drainage ditch with concrete lining.

3 Monitoring and slope remediation

Main design has prescribed periodical monitoring of motorway by geotechnical designer and the motorway maintenance service. Design demanded monitoring during exploatation and to the inception reports. In the inception report defects were detected and quantified and also remediation measures were estimated with their priority order. Based on the report, the Investor ordered additional displacement measurement and detailed remediation design. On Straža Jug location during the visual observation the cracks in the shotcrete were detected on the northern part of the cut. It is peripheral part of the cut were slope changes inclination from 2:1 to 1:1.5. Crack was detected on the slope about 8.3 m from the beginning of the bridge. It was a vertical crack formed in the full height of the first slope floor. Crack width extended from 1.0 cm at slope toe to 5.0 cm at 10 m height. In period from autumn 2010 till spring 2011 crack displacement measurements were conducted. Measurements showed that crack was spreading and developing new cracks. This damage progression formed unstable blocks of shotcrete in size of a few square meters with potential risk for the motorway traffic. Defects were detected on the peripheral part of cut and eroded material from upper slope was deposited on the first bench. Also some material was carried out with water underneath of shotcrete in slope toe. Shotcrete has lost its slope protection function and was covering slope sliding like a mask. Monitoring results didn't find global stability problem, but local surface erosion and a problem of contact between slope and shotcrete was confirmed. At the end of the winter period displacement progress decreased. Decision to start with the remediation measures was made because of the threat of shotcrete blocks to fall down on the motorway.

Figure 2 Defects and crack on Straža jug cut

Visual observation at Gornji Macelj 4 cut detected crack on the contact of concrete stone wall and shotcrete. During 2010 crackmeters were installed and displacements on the cracks were measured. Crack was continuing from the lower slope to the first bench and extended parallel to the motorway (Figure 3). Crack width was from 1.0 cm at the bench to 10.0 cm at the slope toe. At the slope toe soil material was flushed out through the crack. It happened because the large amount of water was drained from the slope in this zone. Wet stain on the slope surface was visible and increasing (Figure 3).

Figure 3 Gornji Macelj 4 – installed crackmeter and vertical crack on the slope

Measurement and detailed visual observation determined that bench had 1.0 cm settlement on the side towards the motorway. Figure 3 shows installed crackemeter for measuring displacement on the crack at the contact of two types of slope protection. The crack was expending and a measurement on installed crackmeters showed that slope with shotcrete protection was moving towards the motorway. This part of the slope had more than 5 mm displacement. Measured displacements and affected slope area indicated that defect was seriously impacting the road safety. The result of the monitoring was the decision to start slope remediation.

4 Mitigation and remediation

Interactive geotechnical design approach [1] prescribes monitoring in design and it is integral part of design process. Regular observation and monitoring during exploatation enabled prompt defect detection and effective remediation. Through interactive geotechnical design slope protection was modified during construction based on the monitoring. Visual observation and measurements gave data about actual material behaviour and the slope protection measures. Local and global instabilities occurred along the motorway section at the beginning of the slope excavation and also during the exploatation. On some parts during design some modifications of solutions were predicted [2]. Additional geotechnical investigations with engineering-geological mapping of excavated slopes and geotechnical measurements were performed on critical locations. For these detected instabilities and with the results of additional investigation works numerical analyses were conducted and new protection measures were applied. Since this two described locations were spotted by applying monitoring system, the idea was developed to analyse mitigation of impact on traffic safety, people and property. Mitigation of slope impact on people and property safety is presented by 4 measures [3] and described in Figure 4. First measure is “Altering”. On the slope this measure can be achieved by peripheral channels for collecting rainwater and with the drainage. Next measure is “Averting” and uses constructions like retaining walls or rockfall protection barriers for redirecting the impact and for people protection. Third mitigation measure is “Adapting”. Adapting is used when risk is pre-identified and solution can be modified and upgraded to protect people and property. Last measure, “Avoiding”, is applied in a way to prevent or to limit the access to the risk area and to avoid or to limit construction. This measure is suitable in the conceptual design phase and through the design phase. In described case when defect occurred in exploatation, one of first three measures can be used. For remediation of cuts Gornji Macelj 4 and Straža jug optimal remediation measure was “Adapting”. Existing slope protection solutions were used, but differences observed during monitoring were analysed and final protection measures were adapted accordingly.

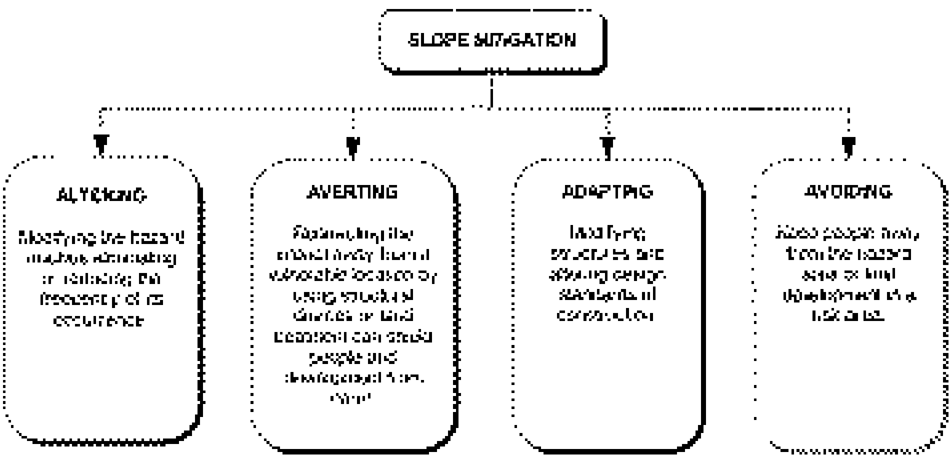

Figure 4 Slope mitigation

At Straža jug cut first was removed cracked shotcrete and eroded material from the slope. In slope toe trench was excavated to the solid ground for the concrete foundation beam which supports renewed slope protection. Four rows of IBO anchors were installed at an angle of 15 ° from the horizontal and at 3.0x3.0 m grid. In upper two rows anchors with a length of 6.0 m and 3.0 m in a lower two rows were installed. The whole slope surface prepared for remediation was covered with 10 cm thick shotcrete reinforced with steel fabric mesh. Existing clogged perforated pipes were protected with geotextile during shotcreting.
At Gornji Macelj 4 cut remediation covered 5 m of stone concrete wall for overlaping and through all area underneath a horizontal crack on the top of the first slope floor. Figure 5 presents a typical cross-section of slope remediation. 4 new rows of IBO anchors were installed at an

angle of 15° from the horizontal and generally at grid 3.0 × 3.0 m. In upper two rows are installed anchor with a length of 9.0 m and 6.0 m in a lower two rows. The whole slope surface prepared for remediation was covered with 5 cm thick shotcrete reinforced with steel fabric mesh. Existing clogged perforated pipes were kept and protected during shotcreting and few extra were installed on places with detected water leaking.

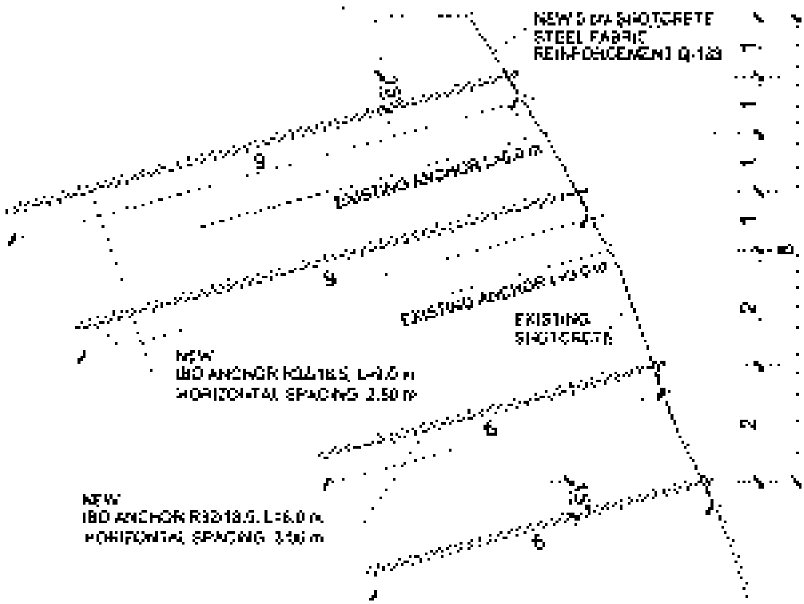

Figure 5 Typical cross-section of slope remediation at cut Gornji Macelj 4

5 Conclusion

This paper shows slope maintenance methodology at the Zagreb-Macelj motorway based on the interactive design approach. This methodology is suitable for linear structures, which in design phase are never ideally investigated and enables investor to seek for economically optimal solutions. The main design analyses the least favourable locations and provides clear guidelines for the construction phase and later exploatation. Continuous monitoring enables modification of solution and system optimization. This approach has benefit in economically favourable and faster construction, but demands planned and continuous maintenance and puts higher pressure on the designer. The final contribution for the user is constant level of safety for the structure, people and property achieved through continuous monitoring and remediation of weakened zones.

References

[1] Kovačević, M.S.: The observational Method and the Use of Geotechnical Measurements. In Proc. 13th Europ. Conf. on Soil Mech. and Geotech. Eng.; 2003; Prague. pp. 575-582.

[2] Humphries, R., Lister, D., Schlotfeldt, P.: Slope Redesign and Slope Remediation during Design/ Build Construction at Kicking Horse Canyon– Phase 2. In Annual Conference of the Transportation Association of Canada ; 2009; Vancouver, British Columbia.

[3] Litz, T.: ESS315 Slope Remediation.; 2010 [cited 2010 March 05. Available from: http://faculty.washington.edu/tswanson/ESS/315/Student%20PP%20Presentations/sloperemediation.ppt.

MULTIPLE LOAD CASE ON FLEXIBLE SHALLOW LANDSLIDE BARRIERS – MUDSLIDE AND ROCKFALL

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Abstract

Shallow landslides distinguish themselves with small failure depths or equivalently small volumes (up to 2 m or 200 m^3 respectively). They release on steep slopes during intense rainfall and runout distances are usually smaller than a hundred meters. However due to their high bulk density and speed, shallow landslides represent a serious hazard to people, buildings and infrastructures such as roads or railway lines. The potential of damage is materialised through the pressure that they exert on objects during impact. The impact pressure is dependent on the kinetics and material properties of the flow as well as the geometry of the flow-object pair. Systematic study of full-scale shallow landslides was carried out at the Veltheim test site in Switzerland. Debris mixtures with volumes of 50 m^3 were released down on 30° degrees steep and 40 m long slope on the hillside of a disused quarry. Mean front velocities between 5 and 11 m/s were measured, while ranged between 0.3 and 1.0 m. Thirty meters downstream of the release mechanism two square-shape obstacles with 12x20 cm sides were installed perpendicular to the flow. Strain-gauge sensors built in the obstacles recorded the forces experienced during the impact. In addition to the pressure measurements, distance sensors hung above the test slope measured flow depths over time and permitted the computation of flow surface velocities through a cross-correlation method. Along steep slopes often besides of shallow landslides, rockfall play an important hazard. Shallow landslide release zones are often caused by a bedrock layer underneath a loose vegetation cover. If the vegetation cover is already eroded or the bedrock layer is rising suddenly the poor rock is on the surface which leads sooner or later to rockfall problems besides the sliding hazard. Both load cases, impact pressures caused by landslides and rockfall impact can be modelled with our finite element software FARO which was originally developed for flexible rockfall barriers and after a certain research time adapted for impact pressure loads. Now suitable protection measures for both hazard can be provided to propose a save solution. Even small snow slides or snow gliding impact which act similar to shallow landslides can be retained within one barrier system.

Keywords: rockfall, shallow landslides, multiple load case, barrier, debris

1 Introduction

Multiple loading on protection measures are a common topic in flexible barrier design. For example, flexible snow nets installed in steep release zone often incur rockfall impacts in summertime, and hence there is a desire both from the customer and designer to combine several load cases within one protection system (Figure 1).

Figure 1 Snow net barrier loaded with rockfall (left), rockfall barrier loaded with a shallow landslide (right)

There are a number of possibilities to meet the design task of a flexible barrier suited to multiple load cases. One approach would be to conduct full scale tests for each of the expected hazard the barrier would face (i.e. rockfall, snow slides, mudslides or tree hit, shallow landslide and debris flow), however this would prove highly costly. A more measured approach is to assess the most important hazards for the design and test for these. Applicable to the case study in this paper the most severe and contrasting hazards were shallow landslides and rockfall.
In this paper rockfall and shallow landside hazards are briefly classified and the important features of their load case are discussed. The approach to testing, modeling and designing a shallow landslides barrier are then presented. Following this the results from rockfall impact testing of the specially developed shallow landslide barriers are examined. The results are then discussed in the context of the multiple load case project for which the barrier system was designed. The paper concludes with a summary of the findings highlighting the requirement to consider multiple load cases in barrier design.

2 Shallow landslide process and flexible barrier development

2.1 Shallow landslide process

Among the large family of landslides, shallow landslides or hill slope debris flows are sparsely documented and studied [1]. They distinguish themselves with small failure depths or equivalently small volumes (up to 2 m or 200 m^3 respectively). They release on steep slopes during intense rainfall and despite their small size they can reach high velocities. The failure process is fast and their location mostly unpredictable. Their evolution from failure to deposition is in the order of dozens of seconds which makes observing their behavior particularly difficult. Runout distances are usually smaller than a hundred meters, while if the slide is channelised by the terrain and is able to entrain material, runout distance may be multiplied several times and the flow will be regarded as a debris flow [2].
Due to their high bulk density and speed, shallow landslides represent a serious hazard to people, buildings and infrastructures such as roads or railway lines. The potential of damage is materialized through the pressure they exert on objects during impact. The impact pressure results from either stopping or deflecting of flowing material encountered by an object. The magnitude and duration of the pressure is dependent on the kinetics and material properties of the flow as well as the configuration of the flow-impact object. Relationships defining impact pressure as a function of the aforementioned parameters are of great importance in mitigation studies delimitating hazard zones and in the design of protection measures like reinforced buildings, retaining walls or flexible barriers [1].

2.2 Shallow landslide barrier development

During shallow landslide testing debris mixtures up to 50 m^3 were released down a 40 m long 30° slope, which reached on average 5 – 11 m/s. It was during these experiments that a flexible wire shallow landslides barrier could be developed to withstand shallow landslide impact pressures up to 200 kN/m2 and also their limit loading capacity could be observed.
All 20 experiments which were conducted permitted a detailed characterization of the flows, in addition to deriving a relationship between flow parameters and impact pressures. Impact pressures in the flow were measured with two square obstacles (12 by 20 cm) fitted with strain gauges placed approximately 10 m before the impact with the barrier. Flow heights and surface velocities were measured with laser distance sensors hung above the flow.
From the load cells installed in the barrier's support ropes it was possible to measure the rope forces required to develop a standard design for shallow landslide barriers. This in addition to shallow landslide flow characteristics from the 20 experiments performed it was possible to develop a fluid structure interaction barrier design model, coupling both an open FOARM flow model and a FARO finite element barrier model ([3], Figure 2). Additionally, an engineered based quasi static pressure design model for the flexible shallow landslide barriers could be developed [3].

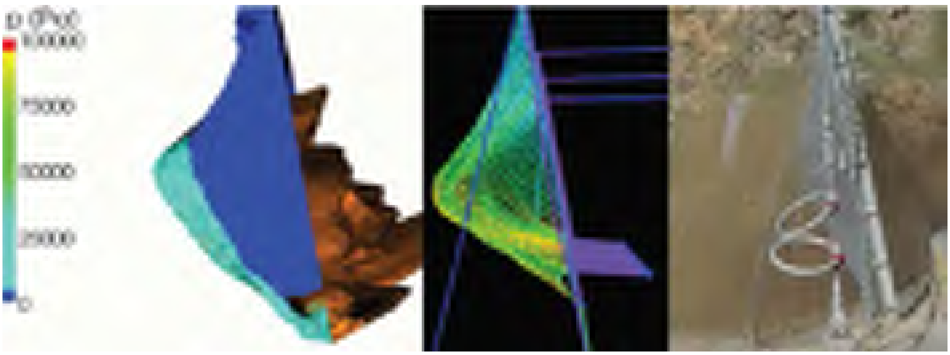

Figure 2 Modeled shallow landslide impact to the flexible barrier with coupled OPEN FOARM software (fluid code) with FARO software (Finite Element Software for barrier design) [4]

From the testing an modeling, two standard barrier systems small shallow landslide with impacts up to 100 kN/m^2 and a stronger one designed for impact pressures up to 150 kN/m^2 could be designed. Higher impact pressures up to 200 kN/m^2 could be achieved only once due to the limitations of the test facility. However, with the aid of the calibrated computer simulation model a design systems for higher pressures shallow landslides could be realized.

3 Rockfall process and flexible rockfall barriers

3.1 Rockfalls

Rockfalls are initiated by detachment of rock debris from cliffs or rock-walls with volumes between 10^{-2} and 10^2 m^3 which enter down-slope motion under the influence of gravity [5]. Rock mass instabilities resulting in rockfall are common along natural rock-cliffs and engineered rock-cuts, and can pose a sever threat to settlement and infrastructure situated beneath. The release mechanism, shape and sizes of detachable rocks are governed by failure along joint planes or discontinuities [6], and detachment is primarily driven by the weathering and erosion acting upon the rock-mass. Following release, rockfall motion consists of falling, bouncing, rolling or sliding [7], the combination of these modes of motion defines the runout path and hazard intensity of the area inundated with rockfalls [8]. Typical propagation speeds can be on the order of 25 m/s; jump heights can reach 20 m or more. This is significant because it gives rockfalls substantial damage potential and it is understandable why steps are taken to predict their runout dynamics and dimension protection structures to mitigate the hazard the pose.

3.2 Flexible rockfall barriers

Rockfall protection structures design spans simple fences to massive earthen dams capable of protecting against rockfall impacts between 100 and 50,000 kJ [9] respectively. On the spectrum of rockfall protection solutions, flexible rockfall barriers are generally designed to deal with impacts in the range of 500 – 5,000 kJ, while flexible rockfall barriers have been designed to withstand impacts up to 8,000 kJ [10]. A key aspect to their design is the flexibility built into the netting and special break elements which extend during an impact and enable the high impulsive and punctual forces of a rockfall to be absorbed over a greater distance reducing the peak forces that act on the barrier. None the less the forces involved during a rockfall are instantaneous relative to a shallow landslide impact and if a barrier is to deal with both these contrasting impacts, adaptations to the barrier system must be made.

4 Rockfall test on standard shallow landslide barriers

To address the differences in load case, the shallow landslides barrier system developed during the testing in Veltheim was tested under the punctual impact load case of rockfall at the Dynamic Test Centre in Vauffelin, Switzerland. The test setup consisted of a 500 kJ horizontal rock impact into the standard SL 150 barrier (impact pressure strength of 150 kN/m^2). The shallow landslide barrier was able to absorb the 500 kJ impact and remained well within its serviceable condition. Tests up to its maximum design capacity of the components were never performed (see Figure 3). However it is expected that the standard system can absorb rockfall energies between 1000 – 1500 kJ with limited damages. It was observed that for higher rockfall impact energies that a few structural system modifications would be necessary.

Figure 3 500 kJ rockfall impact into a standard SL 150 shallow landslide barrier

Comparing rope forces of 500 kJ rockfall impact with forces measured during the shallow landslide impacts the most important differences are the following:

- impact of rockfall is highly impulsive visible on the recorded rope forces with 200-300 ms compared to slide impact which last over 4-5 s;
- peak rope forces similar for both load cases in the support cables;
- peak rope forces in the retaining cables are, however, higher for shallow landslides because each retaining cable due to the accumulated static load of material that piles up behind the barrier as it fills to the full height;
- pressure force in the posts are higher for shallow landslide impacts compared to rockfall impact. We could see plastic deformation at post base plate after area loaded landslide impact. No plastic deformation at the post foundation was visible at the rockfall tests.

These important results have been crucial in making the barrier adaptations for multiple load case rockfall to shallow landslide barriers. The details of the design adaptations are discussed in the following section on the Balisberg project which was the main motivation to perform testing on the barrier for both hazard cases.

5 Project Balisberg – Multiple load case designed rockfall barrier

The Balisberg project was commissioned by the Swiss railway and presented a multiple hazard case involving shallow landslides and rockfalls which threatened the safety of the railway line. We were charged with the task of developing a flexible barrier system to withstand both hazard cases according to the hazard assessment provided by a third party engineering office. Hazard maps were provided for shallow landslide and rockfall in which the intensity, return period and inundated area were delineated (Figures 4 and 5). Both hazard maps give a similar intensity and probability of return period.

Figure 4 100 year return period intensity map of shallow landside shows for the area of projected nets a middle intensity

Figure 5 Intensity map of rockfall for 100 year return period shows at the area of projected nets a high intensity

Middle shallow landslide intensity means slight break failure depth M between 0.5 < M < 2 m and flow height h up to 1 m. From the landslide working group suggestion AGN [11] impact pressures up to 60 kN/m^2 have to be considered for middle intensity. The rockfall hazard map gives rockfall energies with high intensity which means energies larger than 300 kJ (see Table 1). From rockfall field investigation and simulation results the design energy results in 2'000 kJ for the net impact.

Table 1 Intensity classification of rockfall and shallow landslide hazard maps in Switzerland acc. to AGN [11].

Process	Weak intensity	Middle intensity	High intensity
Rockfall	E < 30 kJ	30 < E < 300 kJ	E > 300 kJ
Shallow landslide	M < 0.5 m	0.5 m < M < 2 m h < 1 m	M > 2 m h > 1 m
M = failure depth; h = deposition height of the material; E = Rockfall energy			

5.1 Numerical simulation and barrier adaptation

The required barrier system to meet the hazard case was a combination of a standard SL 100 (design pressure of 100 kN/m^2) system, which would fit for the landslide impact with design pressure of 60 kN/m^2, and a rockfall barrier designed for energies up to 2'000 kJ. This required that the barrier design be verified for this particular load case since these were not the conditions tested during the full scale experiments. The numerical models developed during the full scale testing facilitated the verification and adaptation of the barrier design.

The approach was to take a standard RXI-200 rockfall barrier certified under the Swiss rockfall testing guideline [12] for impacts up to 2'000 kJ to fulfill the rockfall requirements acc. to the rockfall design energy, and expose it to the impact pressures of the expected shallow landslide using FARO (Figure 6.) numerical simulation models. Fig. 6 shows the first impact pressure of the landslide with 60 kN/m^2 dynamic impact over a flow height of 1 m acc. to the hazard map. On the right side the complete filled up barrier is modeled loaded by the hydrostatic pressure of the mud material over the barrier height of 4 m.

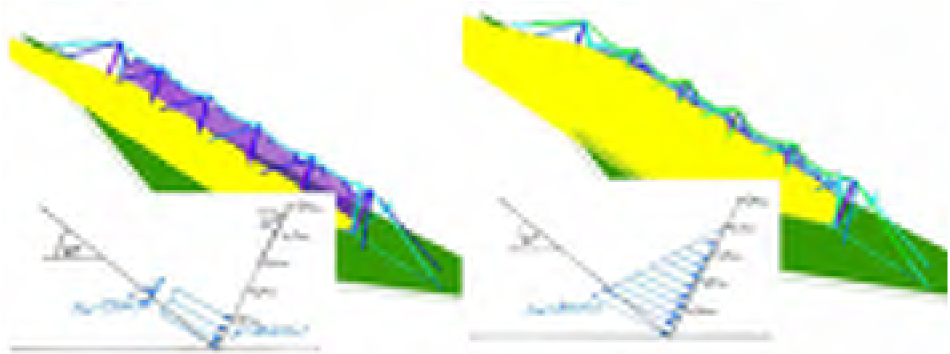

Figure 6 First load case for rockfall barrier dynamic impact pressure of 60 kN/m2 acting over flow height of 1 m (left) and second load case of hydrostatic pressure acting over filling height of 4 m (right)

Full details of the results of the simulation are given in internal design report for SBB [13]. Most important adaptations to the rockfall barrier meeting the addition shallow landslides load were to strengthen the retaining ropes and their anchors, in addition to strengthening the lateral and vertical ropes and selecting a stronger post profile and foundation. All these required adaptation according to the simulation results did not alter the functional capacity of the tested rockfall barrier [14]. That was and important design criteria that had to be proven in the simulations.

6 Conclusion

Through this work it has been proven that flexible wire protection barriers can be designed for greatly differing impact load cases of shallow landslides and rockfalls and has clearly illustrated the case for their requirement. Within the paper two procedures to combine rockfall load with shallow landslide impact pressure have been explained in detail. The first method was based on full scale tests of a shallow landslides barrier system which was additionally exposed to rockfall impacts. This demonstrated how the retaining, lateral and vertical ropes, along with the posts and post foundations of a standard rockfall barrier have to be strengthened to withstand the accumulative pressure loads of shallow landslides. It was found that the area loads of shallow landslides lead to higher forces to the posts due to the spreading behavior of the impact load across all barrier fields. Consequently also higher forces goes to the retaining ropes because of higher slope parallel force component resulting from impact pressure of landslides. Rockfall as highly dynamic impact results in shorter force transmission to the ropes in milliseconds compared to impact loads of shallow landslide filling which can last several seconds. These results can be also transferred to rockfall barriers impacted by creeping snow or small snow slide impacts [15].
Summarized each load case from a different natural hazard has to be considered separately in the calculation but similarities between spread out area loads like snow pressures and landslide and debris flow pressures are obvious for the general loading behavior but of course pressure values vary.

References

[1] Rickli, C. and Bücher, H.: (2005) Hangmuren ausgelöst durch die Unwetter vom 15.-16.07.2002 im Napfgebiet und vom 31.08-1.09.2002 im Gebiet Appenzell – Projektbericht zuhanden des Bundesamtes für Wasser und Geologie BWG.

[2] Bugnion, L. and Wendeler, C.: (2011) Impact pressure measurements in shallow landslides, Proceedings of the Second World Landslide Forum 3-7 October 2011, Rome.

[3] Bugnion, L., Bötticher, A. and Wendeler, C.: (2012) Large scale field Testing of hillslope debris flows resulting in The Design of Flexible Protection Barriers, Proceedings of 12th Interpraevent Grenoble/ France.

[4] Böticher, A.: (2012) Flexible Hangmurenbarrieren: Eine numerische Modelierung des Tragwerks, der Hangmure und der Fluid-Struktur-Interaktion, Dissertation TU-München, Deutschland.

[5] Rochet, L.: (1987) Development of numerical models for the analysis of propagation of rock-falls. 6th Int. Congress on Rock Mechanics, 1, pp. 479-484.

[6] Jaboyedoff, M., et al: (2011) Slope tectonics. Geological Society London.

[7] Dorren, L. K. A.: (2003) A review of rockfall mechanics and modelling approaches, Progress in Physical Geography, 27, 1, pp. 69-87.

[8] Baillifard, F., Jaboyedoff, M., and Sartori, M.: (2003) Rockfall hazard mapping along a mountainous road in Switzerland using a gis based parameter rating approach. Natural Hazards and Earth System Sciences, 3, pp. 431-438.

[9] ASTRA (1998) Bereich Kunstbauten, Planung, Bau und Unterhalt von Schutzgalerien gegen Steinschlag – und Lawineneinwirkunggen. Bern, Switzerland

[10] Geobrugg AG (2012): GBE-Steinschlag-Barrieren 100 kJ – 8000 kJ: Die wirtschaftliche Lösung ihrer Sicherheitsprobleme, Romanshorn, Schweiz.

[11] Bolliger, D., Keusen, H. R., Rovina, H., Wildberger, A., and Wyss, R.: (2004) Report on hazard classification of slides: Permanent slides, spontaneous slides and shallow landslides, Zollighofen Switzerland

[12] Gerber, W.: (2001) Richtlinie über die Typenprüfung von Schutznetzen gegen Steinschlag, Bundesamt für Umwelt, Wald und Landschaft (BUWAL) und der Eidg. Forschungsanstalt WSL, Bern Schweiz.

[13] Wendeler, C. and Neumann, P.: (2013) Simulation RXI-200 auf Hangmuren für SBB, Bericht Geobrugg, Schweiz.

[14] WSL Eidg. Forschungsanstalt (2004) Prüfbericht 04-7 Bericht über die Typenprüfung von Schutznetzen gegen Steinschlag, Eidg. Forschungsanstalt WSL, Schweiz.

[15] Margeth, S.: (2006) Schlussbericht zu alpS Projekt B 2.1 Steinschlagschutzbauwerke unter statischer und dynamischer Belastung von Schnee, Schneerutschen und Kleinlawinen, SLF Davos, Switzerland.

DESIGN OF RAILWAY TRACKBEDS WITH GEOCELLS

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Abstract

Geosynthetics in the form of geotextiles and geogrids made of polymeric materials are being used to improve the bearing capacity of railway trackbeds. These materials provide a confinement effect through friction. In the same manner, geocells refer to a synthetic, honeycomb-like cellular material, the structure of which is interconnected by joints to form a cellular network used for the confinement of soils. A literature survey reveals that the introduction of a 200-mm-high geocell into the upper subgrade layer increases the resilient modulus of this reinforced layer by an average multiplier factor, termed MIF (Modulus Improvement Factor), of 2.5. The MIF is supported both by in-situ FWD and by pressure-cell testing. Empirical calculations, furthermore, indicate that MIF is a function of the properties of the geocell. Thus, this reinforced layer can be regarded as part of a railway trackbed structure. Given these findings, the paper suggests the use of an equivalency procedure in order to calculate the effective thickness of a 200-mm reinforced subgrade layer in terms of the thickness of type A (CBR=60%) sub-base material in railway trackbed structures. For example, when a given subgrade with a CBR value of 10% is reinforced by the 200-mm height of a high-standard geocell, this reinforced subgrade can substitute for a 150-mm type A sub-base layer. Finally, it should be pointed out that (a) the proposed equivalency procedure is valid for a subgrade CBR of up to 12%, and (b) the use of geocell reinforcement should be accompanied by a strictly detailed QA plan for in-situ density and modulus (plate-load testing) of the reinforced layer.

Keywords: CBR, confinement, equivalency factor, geocell, Modulus Improvement Factor (MIF), Railway-trackbed, resilient modulus

1 Introduction

With increased use and development of transportation facilities in Israel, railway trackbeds need to be stable, with no excessive deformation under load, in addition to satisfying the tolerable criteria in terms of load repetitions for fatigue and rutting mechanisms. Often, when carriage loads are increased, there is the possibility of plastic deformation owing to the absence of confinement in the lateral direction when vertical loads are applied. Thus, the effects of confinement on the performance of the railway trackbeds are significant. In the absence of proper confinement, failures in these beds are likely to occur.
Geosynthetics in the form of geotextiles and geogrids made of polymeric materials are being used to improve the bearing capacity of railway trackbeds. These materials provide a confinement effect through friction. In the same manner, geocells, which are a three-dimensional form of geosynthetic materials with interconnected cells filled with soil, have many important advantages when used in railway trackbeds (see Fig. 1). In more detail, a geocell refers to a synthetic, honeycomb-like cellular material; a structure of these cells interconnected by joints to form a cellular network is used for the confinement of soils.

Figure 1 Typical geocell mattress, after [1]

In Israel, the present design guidelines for railway trackbeds given by Livneh et al. in [2] do not include the use of geocells. Thus, it seems necessary to include this use in the design guidelines. In light of all the above, the objectives of this paper are as follows:
· conducting a literature review of the increase of the resilient modulus as a result of introducing geocell enforcement into a given layer;
· examining the values of the Modulus Improvement Factor (MIF) through empirical equations to display the influence of the geocell properties on these values;
· developing equivalency factors for the geocell-reinforced layer as a function of (a) the MIF value and (b) the CBR value of the given layer prior to reinforcement.

The sections to follow will detail the process of attaining this paper's three objectives and present associated conclusions.

2 Literature review of resilient modulus increase

Al-Qadi and Hughes [3] conducted field studies to evaluate the use of geocells in flexible pavements. The researchers selected the reconstruction of a road that showed excessive rutting. The use of geocells was chosen as the solution on an experimental basis, and the results pointed to the fact that the pavement laid on the confined base showed no signs of rutting. Unfortunately, it was difficult in all these cases to isolate the effect of the geocell-confinement system as has been used in combination with geogrid, geotextile, or both. However, it can be concluded that in sections where 100-mm-thick geocells were used, the resilient modulus of the aggregate layer increased almost twofold owing to the material's confinement—i.e., MIF=2.0. As a result of the aggregate confinement provided by the geocell and the subgrade separation from the sub-base provided by the geotextile, it appears that a geotextile-geocell combination may provide a significant improvement to overall stability when used on top of a weak subgrade of a heavily trafficked pavement.
Emersleben and Meyer [4] conducted large-scale model tests and field tests, and these showed similar results to [1], which verified the fact that geocells reduce surface deflections and vertical pressure on the subgrade. The tests also studied the effect of aspect ratio, and these results demonstrated that performance improved as the height-to-diameter ratio was increased. At the end of the study period, it was found that the use of geocells not only reduced the material required, but also improved the speed of construction. Along with these field tests, Emersleben and Meyer conducted large scale model tests in test boxes measuring 2m x 2m x 2m. Those tests showed that surface deflection was less in a geocell-confined section. The results were verified by falling weight deflectometer (FWD) measurements carried out in field studies. More specifically, compared to an unreinforced test section, the stresses beneath the geocell layer were reduced by about 30 percent. In addition, the FWD results showed that back-calculated layer modules in the first test section were 290 MPa for 400 mm gravel and

320 MPa for 200 mm gravel plus 200 mm geocell. In the second test section, the back-calculated values were 350 MPa and 450 MPa, respectively. With the aid of these values, it can be shown that the resilient modulus of the gravel layer increased for the first test section by only MIF=1.2, and for the second test section by MIF=1.6, both because of the gravel confinement.
Rajagopal and Kief [5] argued that results of studies demonstrated that the unique interaction of soil, cell, and shape in cellular confinement systems acted to stiffen pavement foundations as a result of the soil-confinement mechanism. Their paper describes a case study, together with the in-situ testing details, and the analysis and explanation of the structural contribution of a three-dimensional cellular confinement system on soft soil. The authors came to the conclusion that the resilient modulus of the gravel layer increased by MIF=5.0, from 100 MPa to 500 MPa, because of the gravel-confinement effect as expressed in the readings of the installed pressure cells. This conclusion is derived for a case in which the loading has been induced directly on the surface of the granular reinforced layer, and not for the case of real structures (i.e., on top of additional structural layers covering the reinforcement layer). Again, note should be made that the conclusion was not derived from FWD measurements, but from cell readings. For this type of loading, the installed pressure cells showed, that compared to an unreinforced section, the stresses beneath the geocell layer were reduced by about 51 percent.
Hegde and Sitharam [6] indicated the beneficial effect of geocell reinforcement in soft clay beds through 1-g model plate load tests and numerical simulations using FLAC2D. Results showed that the provision of geocells leads to a fivefold increase in the load- carrying capacity of a very soft clay bed. This impressive finding, however, is limited to cases in which only plasticity failure takes place. Thus, for cases in which the theory of elasticity holds (such as the design of a railway trackbed), this finding is not applicable. The paper also revealed that the overall performance of the very soft clay bed improves further because of the provision of planar geogrid at the base of the geocell. Numerical results were also in line with the experimental findings.
In contrast to the aforementioned plasticity case, Zang et al. [7] showed for the elasticity case that by confining the upper 200 mm of the soil surface with geocell, it can be assumed, based upon laboratory and field-test results, that the reduction in maximum vertical stress is about 35%. This finding led Kief [8] to obtain an increase in the resilient modulus by MIF=4.7, together with an unexplained transition zone beneath the reinforced layer that possessed a 1.5-time increase in resilient modulus. Here, it must be noted that Reference [8] also objects to the running of FWD measurements for exploring the rate of increase in the resilient modulus resulting from the geocell confinement effect. This objection, however, is not compatible with the use of FWD measurements in [3] and [4].
Kief [8] showed that the resilient modulus of the gravel layer increased by MIF=2.4, from 420 MPa to 1,010 MPa, because of the gravel-confinement effect. As in [5], this conclusion is derived from pressure-cell readings (and not from FWD measurements) for the in which the loading has been induced directly on the surface of the granular reinforced layer, and not for the case of real structures; i.e., on top of additional structural layers covering the reinforcement layer.
To sum up, the range of the multiplier increase in the resilient modulus of a given layer because of geocell reinforcement varies between MIF=1.2 and MIF=5.0 according to the aforementioned findings. Kief [14] makes almost the same statement that MIF varies between 1.5 and 5.0. The upper values of this range, however, are rather questionable, as no FWD measurements have been conducted to prove the existence of these upper values. Here it is important to note that according to Han [15], FWD measurements utilize too small deformations to mobilize geosynthetic to be effective. Thus, this method is incapable of detecting the benefit of geosynthetic reinforcement. If this last statement is true, one may question Han's [15] contention that this method is capable of detecting the above-mentioned benefit for trafficked pavements if a control section is available. FWD measurements, then, cannot be ruled out for pavements containing geosynthetic reinforced layers.

3 Geocell reinforcement equations

Rajagopal et al. [9] proposed the following equation for the layer modulus of geocell-confined granular material in terms of the secant modulus of the geocell material (M) and the modulus number of the unreinforced sand (Ku):

$$E_G = 4 \times (\sigma_3)^{0.7} \times \left(Ku + 200 \times M^{0.16}\right) \tag{1}$$

where:

E_G layer modulus of geocell-confined granular material, in kPa;
Ku modulus number for unreinforced granular material as defined by Duncan and Chang in [10];
M secant modulus of geocell material, in kN/m;
σ_3 confining pressure, in kPa.

This equation is based on the older model of the dependency of granular modulus on confining pressure. However, newer equations for the granular modulus exist in the technical literature [16]. Also, the additional confining pressure owing to the membrane stresses can be calculated using the following equation given by Henkel and Gilbert in [11]:

$$\Delta\sigma_3 = 2 \times M / D_o \times \left[1 - (1 - \varepsilon_a)^{0.5}\right] / (1 - \varepsilon_a) \tag{2}$$

where:

$\Delta\sigma_3$ increase in the lateral pressure base on the membrane correction theory, in kPa;
D_o initial diameter of the geocell, in meter;
ε_a axial strain of the geocell.

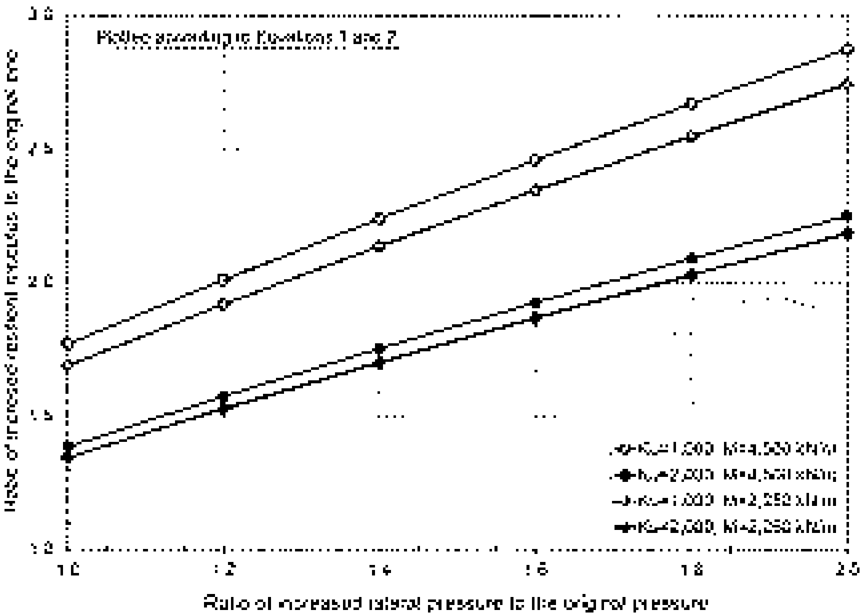

Figure 2 Geocell confinement effect on the resilient modulus as calculated from Eq. (1) and Eq. (2)

In the calculated example of [1], Ku is equal to 1,000 and M=4,500 kN/m. For these values, Fig. 2 shows that the ratio of the increased resilient modulus of the reinforced layer to its original resilient modulus prior to reinforcement varies as a function of the ratio of the increased lateral pressure to its original pressure from MIF=1.8 up to MIF=2.9 (MIF - Modulus Improvement Factor). For the reinforcement of a stiffer granular layer possessing Ku=2,000 with the same geocell possessing M=4,500 kN/m, the above range is lower, starting at MIF=1.4 and then increasing up to MIF=2.2. For the example in [1], the calculated value of the increased resilient modulus of the reinforced layer was 760 MPa. This value seems to be rather high.
As a result of this high value, Fig. 2 includes the two MIF curves calculated for a lower value of M—i.e., 2,250 kN/m. These two curves indicate that their associated MIF are lower than those associated with M=4,500 kN/m.
In addition, Fig. 2 indicates that the resilient modulus of the reinforced granular layer increases with the increase in the value of M—i.e., the secant modulus of the geocell material. Values of M, which are given in [12], vary from 100 kN/m up to 1,000 kN/m. Thus, the use of geocells possessing these low values of M leads to lesser MIF values shown in the figure. To sum up, it seems that for design purposes, the suggested MIF value can be taken as 2.5. This value is supported by both the literature review presented earlier and the geocell reinforcement equations given in the present section.

4 Geocell reinforcement equivalency

The Israeli guidlines for the structural design of railway sub-ballast trackbeds [2] utilizes the equivalency method. In this method, 100 mm of sub-base type A (CBR =60%) are equall to 125 mm of sub-base type B (CBR=40%) or 175 mm of subase type C (CBR=20%). In other words, it can be shown that for 100 mm of sub-base type A (CBR=60%), their eqivelancy in terms other sub-base types possessing any design CBR can be formulated as follows:

$$H_{EQ} = 0.03125 \times CBR^2 - 4.375 \times CBR + 250 \qquad (3)$$

where:

H_{EQ} equivalent thickness of 100 mm of sub-base A in terms of a sub-base with an inferior strength value type, in mm;
CBR design CBR of the inferior sub-base, lower than 60%.

Obviously for CBR=60%, H_{EQ}=100 mm. Now, the suggested method outlined in the present section is based on the fact that the introduction of the geocell reinforcement increases, as mentioned in the two previous sections, the existing resilient modulus of the infill material by a ratio of M. This increase also increases the CBR rate of the reinforced material in the following way:

$$CBR_{IN} = CBR_{EX} \times \left(E_{IN} / E_{EX}\right)^{1.41} = CBR_{EX} \times (MIF)^{1.41} \qquad (4)$$

where:

E_{EX} existing resilient modulus;
E_{IN} reinforcement resilient modulus;
CBR_{EX} existing CBR value;
CBR_{IN} increased CBR value.

At this junction, it is worth noting that Eq. (4) is based on the following equation, taken from [13]:

$$CBR = \left(E/\alpha\right)^{1.41} \tag{5}$$

where:

E	existing resilient modulus of the given mateial;
α	regression coefficient;
CBR	existing CBR value of the given material.

From the material equivalence thickness reported in Eq. (3), and the proper substitutions, it can be shown that CBR_{IN} leads to the following equivalence thickness (H_{EQIN}):

$$H_{EQIN} = 0.03125 \times \left[CBR_{EX} \times (MIF)^{1.41}\right]^2 - 4.375 \times \left[CBR_{EX} \times (MIF)^{1.41}\right] + 250 \tag{6}$$

Here, H_{EQIN} denotes the equivalent thickness of 100 mm of sub-base A in terms of a reinforced subgrade layer with a strength value (prior to the reinforcement) of CBR_{EX}. In other words, a 200-mm reinforced subgrade layer can reduce the subgrade type-A layers in the railway trackbed by ΔH_A, a value expressed in the following expresson:

$$\Delta H_A = 200 \times \left(100 / H_{EQIN}\right) \tag{7}$$

Finally, Fig. 3 depicts the variation in ΔH_A with the increase in CBR_{EX} for the various rates of MIF. As shown at the end of Section 3, the suggested design rate of E_{IN}/E_{EX} is MIF=2.5. For this rate of MIF and CBR_{EX}=10%, the figure shows that ΔHA is equal to 150 mm. Furthermore, it is suggested that the final structure will contain at least a sub-base type-A layer of 200 mm thickness. Obviously the figure allows ΔH_A determinations for other values of CBR_{EX} and MIF.

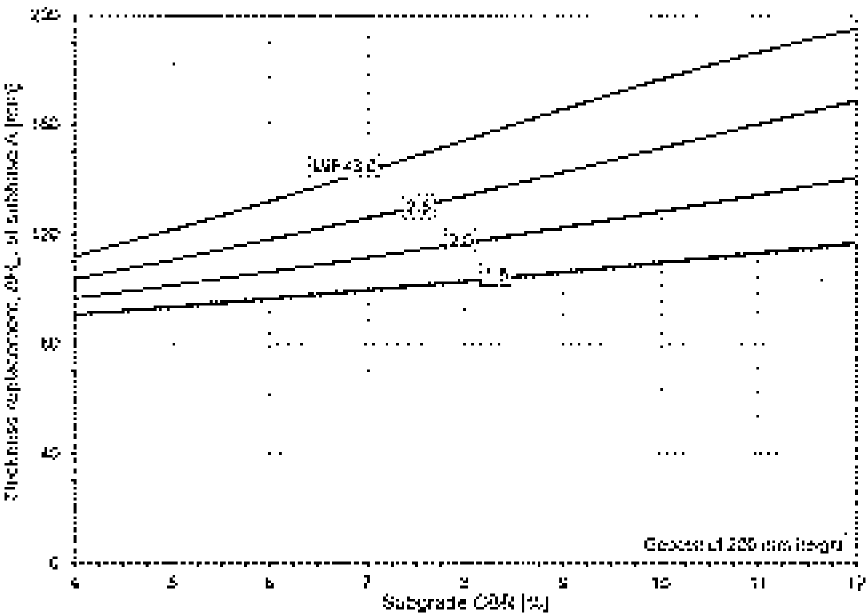

Figure 3 Thickness replacement of sub-base type A as a function of installing Geocell into the subgrade material possessing a given CBR value

To conclude, it should be stated that the present proposed equivalency procedure is valid for a subgrade CBR of up to 12%. Also, beacuse of locally mixed experience, both reasonable and bad, the use of the geocell reinforcement should be accompanied by a strict and approved QC and QA plan for the in-situ density and in-situ modulus of the reinforced layer.

5 Conclusions

This paper dealt with the issue of reinforcing the upper subgrade layer beneath the railway trackbed structure with a net of geocells. The literature review conducted in this paper reveals the increase in the resilient modulus of the reinforced layer as a result of the geocell reinforcement. The increase in the resilient modulus described is expressed by a multiplier factor, termed MIF (Modulus Improvement Factor). According to the literature, MIF varies between 1.2 and 5.0. The upper values of this range, however, are rather questionable, as no FWD measurements have been executed to prove the existence of these upper values.
Following the literature survey and the geocell reinforcement equations presented in the present paper, it seems that for design purposes, the suggested MIF value can be taken as 2.5. To conclude, this paper has developed the necessary equations for calculating the equivalent thickness of a 200-mm-thick, geocell-reinforced, upper subgrade layer in terms of the thickness of sub-base type A (CBR=60%). It has been shown that this equivalent thickness is a function of the existing subgrade CBR (with a maximum value of 12%) and the MIF rate. For a subgrade CBR of 10% and MIF=2.5, this 200 mm can replace 150 mm of subgrade type A in the railway trackbed structure. Finally, it should be emphasized that as the railway-design method does not allow any thickness reduction for upgrading the sub-base layers from CBR=60% to higher CBR values, no thickness reduction is allowed for reinforcing a sub-base layer with geocells.

References

[1] Babu, S.G.L., and Kumar, P.: An Approach for Evaluation of Use of Geocells in Flexible Pavements, Proceedings of Indian Geotechnical Conference, (Paper No. E502), Delhi, 2012.

[2] Livneh, M., Craus, J., Sloosker, K., and Rabinovitz, A.: Design Methodology for Railway Subballast Trackbeds, Proceedings of the 2006 International Railway Symposium, Ankara-Istanbul, Turkey, 2006.

[3] Al-Qadi, I.L., and Hughes, J.J.: Field Evaluation of Geocell Use in Flexible Pavements, Transportation Research Record 1709, pp. 26-35, 2000.

[4] Emersleben A., and Meyer N.: The Use of Geocells in Road Constructions-Falling Weight Deflectometer and Vertical Stress Measurements, EuroGeo4, Paper No. 132. Proceedings of the 4th European Geosynthetics Conference, Edinburgh, 2008.

[5] Rajagopal, K., and Keif, O.: Three Dimensional Cellular Confinement System— Contribution to Structural Pavement Reinforcement, Geosynthetics India-08, Hyderabad, India, 2008.

[6] Sitharam, T.G., and Hegde, A.: Performance of Shallow Footing on Geocell Reinforced Clay Bed Using Experimental and Numerical Studies, Proceedings of Indian Geotechnical Conference, (Paper No. E512), Delhi, 2012.

[7] Zang, L., Zhao, M., Shi, C., and Zhao, H.: Bearing Capacity of Geocell Reinforcement in Embankment Engineering, Geotextiles and Geomembranes 28, pp. 475-482, 2010.

[8] PRS Mediterranean Ltd.: Construction of a High Diameter Road, St. Petersburg By-pass, Russia, Using Neoweb Reinforcement, Private communication, 2013.

[9] Duncan, J. M., and Chang, C. Y.: Nonlinear Analysis of Stress and Strain in Soils, Journal of the Soil Mechanics and Foundations Division: American Society of Engineers 96 (5), pp. 1629-1653, 1970.

[10] Rajagopal, K., Krishnaswamy, N.R., and Madhavi L.G.: Finite Element Analysis of Embankments Supported on Geocell Layer Using Composite Model-Computer Methods and Advances in Geomechanics, in C. S. Desai, T. Kundu, S. Harpalani, D. Contractor, and J. Kemeny, Eds., Balkema, Rotterdam, the Netherlands, pp. 1251-1254, 2001.

[11] Henkel, D. J., and Gilbert, G. C.: The Effect of Rubber Membranes on the Measured Triaxial Compression Strength of Clay Samples, Geotechnique 3, pp. 20-29, 1952.

[12] Madhavi, L.G., and Rajagopal, K.: Parametric Finite Element Analyses of Geocell-Supported Embankments, Canadian Geotech Journal 44 (8), pp. 917-927, 2007.

[13] Livneh, M.: Correlative Studies of FWD, LDW, and DCP Testing of Railway Track Sub-Ballast Structures in Israel, Proceedings of the 9th International Conference on Bearing Capacity of Roads, Railways, and Airfields (BCRR 2013), Trondheim, Norway, Vol. 1, pp. 209-218, 2013.

[14] Kief, O.: Neoweb® 3D Cellular Confinement System for Structural Pavement Reinforcement of Roads & Railways. Geotec. Hanoi, 2011.

[15] Han, J.: Geosynthetics Recent Research and Development, an Oral Presentation, Long Beach Convention Center, Long Beach. CA, Private communication, 2013.

[16] Uzan, J.: Resilient Characterization of Pavement Materials, International Journal for Numerical and Analytic Methods in Geomechanics, Vol. 16, No. 6, pp. 435-459, 1992.

SUBSOIL STONE FOREST DISCOVERED DURING THE CONSTRUCTION OF THE MOTORWAY (SE SLOVENIA)

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Abstract

The monitoring of the construction of the motorway in SE Slovenia by karst researchers has also proved to be of great value in the exploration of natural heritage through the deepening of knowledge about the formation and development of this part of the Slovenian karst. Characteristic subsoil karst surface was formed under a cover of sediment of varying depth. Subsoil formation of carbonate rock also marks the entire epi-karst and vadose zone. The surface is carved into subsoil stone forests, the surfaces of the karren are smaller and numerous hollow shafts are filled with fine-grained sediment. The outstanding characteristics of the karst surface are primarily the result of large surfaces of stone forests, which are difficult to detect prior to earthwork or geo-physical research. Most of the surface is above the underground water level. There is only one exception of the motorway section, which was not fully uncovered due to the construction method and because lower road beds were put upon a special grounding. For this reason we could only research the karst formations that had been shaped by water percolation through the karst surface. We did not find the characteristic subsoil karren as those discovered during the earthwork along previous sections, which were also formed by the fluctuation of underground water. Karst features discovered during construction work give us insight into the characteristics and manner of formation of the karst in the wider area of south-eastern Slovenia; they thus unveil yet another characteristic of karstic natural heritage that was hidden from view, but which also provides guidelines for planning activities on the surface.

Keywords: stone forest, subsoil formation of carbonate rock, Dolenjska karst, Slovenia

1 Introduction

The monitoring of the construction of the motorway in SE Slovenia by karst researchers has proved to be of great value in the exploration of our natural heritage through the deepening of our knowledge about the formation and development of this part of the Slovenian karst. This characteristic subsoil karst surface was formed under a cover of sediment of varying depth. Subsoil formation of carbonate rock also marks the entire epi-karst and vadose zone. The surface is carved into subsoil stone forests (Figs. 1, 2), the surfaces of the karren are smaller and numerous hollow shafts are filled with fine-grained sediment. The outstanding characteristics of the karst surface are primarily the result of large surfaces of stone forests, which are difficult to detect prior to earthwork or geo-physical research. Most of the surface is above the underground water level. Karst features discovered during construction work give us insight into the characteristics and manner of formation of the karst in the wider area of southern Slovenia; they thus unveil yet another characteristic of our karstic natural heritage that is hidden from view, but which also provides guidelines for planning activities on the surface.

Figure 1 Uncovering of subsoil stone forest

Figure 2 Subsoil shaped wider pillars of stone forest

2 Main characteristics of the landscape

The north-eastern part of the motorway section starts in the vicinity of a swallow hole of the Igmanca stream at characteristic Dolenjska lowland. Lateral, changing and relatively thick beds of sediment and soil cover the land. The underground water is close to the surface of

the predominant Fluviokarst [1]. Here we find individual karstic features, among them minor swallow holes, esatavellas and, to a lesser extent, an outcrop of carbonate rock. We have detected thinner sediment beds and frequent outcropping of karstified rock, which mostly disintegrates into small fragments. There is less surface water at that location since it flows into the subsoil relatively quickly due to the inclination of the terrain, the thin sediment beds and fragmentary cover of disintegrated rock. The stone forests and karren, which reach various depths, are composed of compact and also tectonically very crushed rock. Where the rock is not crushed and where compact blocks of limestone occur between the cracks, we noticed stone teeth on the surface. During the earthwork, these had in many cases revealed themselves as real stone pillars. Where the rock was tectonically cracked or crushed, we did not find karren on the surface; however, at some locations rudimentary stone teeth hidden beneath the soil immediately disintegrated during earthwork. A rare network of streams is formed on the surface, but a substantial part of the source and side channels do not have permanent flows. Minor springs of underground water are frequent and flow along narrow, corroded cracks in the rock. Surface and subsoil karst features are rare. The surrounding valleys are dry for most of the year, and streams and floods only occur after downpours. Percolating water feeds small but permanent sources. Fluctuation of the flow from these sources is minimal and rarely more abundant, which reflects greater permeability and cavernosity. Underground water flow is close to the surface, for which reason karst formations such as this are called “shallow karst”.
Thick beds of Plioquaternary sediments on carbonate rock, especially on a moist surface, are usually acid. It is not yet fully understood whether these sediments are autochthonous or brought from the nearby dolomite surroundings. After comparison with the circumstances in tropical karst, the opinion developed that in these valleys the thick layers of disintegrated material could have been preserved only because of the high underground water level and poor erosion, and thus developed into subsoil karst.

3 Karst features

3.1 Karren surface

Most of the surface of the higher-lying land is karren. The bottoms of dales are covered by sediment beds. Two types of karren can be clearly distinguished. Karren in their original meaning occur in areas that are not covered by sediment, but with thin layer of soil. They cover the major part of the surface. The carbonate rock is dissected along the cracks, and on the rock we can detect the traces of its former a) subsoil formation [2, 3] – these are relatively few, b) and indirectly transformed by precipitation, the surface was overgrown and c) finely shaped by bio-corrosive factors. The surface was mostly forested. Such karren existed on the surface of the cone in the dale at the beginning of the motorway section.
Especially in one area we observed well-expressed bio-corrosive activities on numerous karren outcrops of carbonate rock. The rock was more diluted by bio-corrosion on the shadier sides. Moss mostly grows there while lichen can also be found on areas exposed to the sun. Bio-corrosive processes do not take place equally on the entire surface of the rock, but selectively. Most probably lithologically slightly different clasts in crushed and then cemented rock are diluted to various depths or else there are different organisms in the various neighbouring clasts. The contact areas between the various clasts are especially corroded, in some spots up to several mm deep.
The major part of the forest-covered surface was dissected mainly by individual rocks of various sizes with partially similar traces of formation to the karren described above. They are frequently defined by funnel-shaped mouths of larger subsoil channels. The rocks reach up to one to two metres in height, with a narrowing on top and with large areas of soil between them. The earthwork uncovered them as the tips of the larger areas of stone forests.

3.2 Subsoil stone forest

Relatively large areas of subsoil stone forests illustrate the manner and long-lasting subsoil formation of this part of the karst surface, covered by fine-grained sediment and soil. The stone pillars are completely covered by the sediment and soil, or their tips protruding on the surface. The configuration of the surface includes smaller or larger dolinas. The largest have a diameter of several tens of metres. Some are filled with grey clay, the origins of which we are still investigating. Both are dissected by subsoil stone forests.
The subsoil stone forests are composed of a dense network of more or less thickset and pointed pillars, which reach up to 8, sometimes even 10 metres, although most are lower. The narrower pillars with a diameter of one to two metres have sharp or rounded spire, while the thickest ones reaching up to ten metres (Fig. 3) in width have one or more spires or their tops are composed of more or less curved crests. Among them in most parts are funnel mouths of the perpendicular subsoil channels or horizontal subsoil channels.

Figure 3 Pointed top of narrower pillar

Subsoil rock features predominate in the rock relief of the pillars [2, 3], which indicates gravitational flow of water from the surface. These are mainly subsoil channels. The most typical are the vertical ones (Fig. 4) with diameters that can reach up to one metre, the largest of which, as we shall explain below, can also be called subsoil shafts, which at the top develop into funnel-shaped mouths. Surface water that flows from the soil along the rocks collects in them. In cross-section the funnel-shaped mouths can take various shapes. They can be open, semicircular or nearly round. Their shapes are often the result of the permeability of the rock and the sediment contact along which the water flows downwards. Long, drawn-out formations of the mouth at less permeable connections causes the rock features to erode deeper into the rock. The water found its way through the rock less frequently and veritable funnels were shaped. Smaller and curving channels are formed at less permeable connections or when minor quantities of water collect on the surface. This is characteristic for smaller pillars with tops that project out of the earth. Subsoil scallops are rare and generally indicate a well-permeable rock connection with the sediment that surrounds it, and moreover, we find elongated notches on the pillar walls, which are traces of water accumulation at the less permeable part of the connection and accelerated corrosion of the rock next to it.

Figure 4 Subsoil channel

Close below the surface where the rock is covered with soil the rock surface of the stone pillars is relatively smooth, while deeper, at the connection with the sediment that covers the surface, it is coarse and often has a configuration of rounded pendants. The rock there is weathered. The thickness of the weathered layer measures up to 1 cm. It is soft when moist, but as it dries out after being exposed on the surface for a longer period and the water evaporates from it, it hardens. The state of weathering of the top layer of the rock is the result of the connection with the sediment, which is moist most of the time. The connection is relatively less permeable, and the water that does permeate it only slowly washes the solution. The connection with more permeable soil is correspondingly also more permeable.

3.3 Karst hollows

Uncovered hollows are generally the result of vertical water percolation through the epi-karst and the vadose parts of the aquifer. The shafts can be classified as hollow and those filled with sediment (Fig. 5). The latter are termed subsoil due to the similarity of their features with subsoil rock, mainly with vertical subsoil channels.
The fifteen shafts, vertical and mostly simple with only one in level, the deepest measuring 24 m, three others deeper than 10 m, while the rest were less deep, with diameters reaching up to 5 metres, but in most cases less, indicate here and there, greater permeability, allowing dense vertical water percolation due to their expressed vertical cracks and vertical rock strata. The shafts occur among the subsoil karren and forests. They do not reach the surface. Their walls are carved with larger or minor vertical channels and often covered with a thin layer of sediment, which causes their thiny dissection [2, 3]. The floors of the shafts are often covered by sediment or sediment fills their lower part.
Subsoil shafts are more or less vertical hollows, similar to ordinary shafts, through which water also percolates from the karst surface, but they are almost entirely filled with sediment, with only individual vertical sections hollow. The water that flows through them deposits the sediment that covers the surface. Their cross-sections are more or less round or extended at the cracks and bedding planes. Their diameters reach two metres. Sediment filling facilitates the shaping of their periphery, and notches appear at less permeable connections. Subsoil

shafts are formed at local dense flow of larger quantities of water. They can develop from subsoil channels. Their walls are carved with along-sediment rock features, which are the traces of formation at the connection with fine-grained sediment. With greater permeability in the karst interior, the subsoil shafts can be emptied.

Figure 5 Subsoil shaft along fault

Above-sediment channels often occur on horizontal bedding planes and in cracks, or networks of anastomoses, the traces of paragenetic stratification. Thus, temporarily flooded areas occur locally and the water, which carries fine-grained sediment and deposits, cuts its way upwards.

4 Conclusion

More and more, the unique natural heritage and development of the Dolenjska karst reveal themselves. The importance of the participation of karst researchers in planning major activities in the environment and monitoring the work has been demonstrated once again. Cooperation with the road constructors has set an excellent example for many years.
This time we were given the opportunity to follow the water precipitating into the epi-karst and the upper part of the vadose zone, which were shaped under a relatively thick cover of sediment and soil and where stone forests, shafts and subsoil shafts have been formed over large areas. The scarcity of stone forests and special geomorphological karst features characteristic of this part of the karst [4] demand that we prepare guidelines for further planning of activities in the karst landscape.

References

[1] Gams, I.: Kras v Sloveniji, Založba ZRC, 2004.

[2] Slabe, T.: Subcutaneous rock forms, Acta carsologica 28(2), 255-269, 1999.

[3] Slabe, T. & Knez, M.: Kraške podtalne skalne oblike, Annales 14(2), 259-266, 2004.

[4] Knez, M., Otoničar, B. & Slabe, T.: Subcutaneous stone forest (Trebnje, Central Slovenia), Acta carsologica 32(1), 29-38, 2003.

APPLICATION OF INDUSTRIAL WASTE MATERIALS IN SUSTAINABLE GROUND IMPROVEMENT

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Abstract

Urbanization process inevitably leads to lack of suitable construction surfaces for further development activities. Therefore, locations which were in past neglected, due to their inadequate ground characteristics, are nowadays reconsidered for construction purposes. In order to enhance geotechnical characteristics of ground, various ground improvement techniques have been developed. This paper discusses existing technologies which use man-made or natural materials in ground improvement process. One of the biggest issues, regarding technologies which use cement or lime as binder, is in their environmental impact. To produce one ton of improved soil, grouting and admixture techniques require 20 kg of cement or lime which release 18 kg of CO_2 into atmosphere. Hence, these technologies are environmentally suboptimal and there is a large potential in developing new methods which tend to reduce environmental footprint. This can be achieved by partially replacing standard binders with fly ash, which is residual waste material. Other possibility of achieving sustainability is replacement of standard gravel or crushed rock aggregate in stone columns technique with industrial slag. Application of waste materials in sustainable ground improvement is a rapidly-developing research area. Besides potential environmental benefits, utilization of waste materials would also solve issue of their disposal, what is significant due to large deposit amounts of fly ash and slag in region.

Keywords: ground improvement, sustainability, fly ash, slag, deep soil mixing, vibro-stone columns

1 Introduction

Cities worldwide are rapidly developing and the rate of urbanization is such that by 2050 two-thirds of the world's population will live in urban areas [1]. The urbanization leads to development of industrialization process which brings along a number of negative effects to the environment and on social life. One of such negative effects is production of industrial waste materials, where the process of waste management is very complex and it deals with huge amounts which need to be disposed in a proper manner, with primarily all safety aspects met, following the other aspects (finances, time, etc.). For some time, there is worldwide trend of reusing industrial waste as secondary raw material in many applications. The reason for this is twofold – due to the rapid consumption of natural resources they become more expensive, while on the other hand waste material landfills are becoming larger, and countries are committed to their reuse and recycling. Further, it is inevitable to find solutions for lack of suitable construction surfaces for further development activities caused by urbanization process. Two solutions are possible to overcome the problem of construction site deficiency. First one is expansion of cities in underground where increasing number of underground structures gives rise to the

complexity of underground building systems. Second solution is to 'stay on surface' and to utilize the ground which is, from geotechnical aspect, originally unsuitable for construction.
These two problems of non-satisfactory ground characteristics and accumulation of industrial waste materials on deposits can be dealt through a ground improvement as one of disciplines which tend to reuse waste materials. In second part of 20th century, a series of techniques for engineering treatment of ground were developed in order to enhance its geotechnical characteristics making it suitable for construction. But even though these techniques lead to enhancement of ground characteristics, they sometimes require usage of large amounts of natural resources and can cause, to a lesser or greater extent, pollution of the surrounding environment. Cerić et al. [2] analysed risks of ground improvement and stated that risks may impact time, cost, quality, or the environment. This negative environmental footprint can be manifested not just through pollution of environment, but also through extensive usage of natural resources. Two waste materials, most commonly mentioned in terms of ground improvement, are fly ash and slag, Fig 1.

Figure 1 Industrial waste materials: fly ash (left) and slag (right)

With the aim of responsible waste management Croatian Government has adopted, in 2005, the 'Waste Management Strategy', which is based on reducing the amount of waste by the separation of waste and the promotion of its reusing. However, this concept of waste reuse is not implemented on large scale. Since application of ground improvement techniques is emerging discipline with large number of projects in Croatia, it is interesting to mention some potential positive environmental aspects of using waste materials for ground improvement.

2 Sustainable ground improvement using fly ash

Fly ash is a by-product from combustion of coal in thermal power plants and it became available in the 1930s. Approximately at the same time, a research based on the use of fly ash in concrete started, where shortly thereafter, industry based on fly ash started its ascent when it was used in concrete to build dams. From the point of fly ash application in ground improvement, techniques which use cement and lime to improve the characteristics of the ground are interesting. Production of standard binders, cement and lime, consume large amounts of natural raw materials and fossil fuels. As a result of chemistry, this leads to the release of significant amounts of CO_2 into the atmosphere. Any reduction in the consumption of cement and lime for soil stabilization process so far (according to previous experience of 2-5% by weight of cement stabilized soil) will contribute in reducing the consumption of natural raw materials and fossil fuels and thus reducing CO_2 emissions [3].
Today, dominant application of fly ash is through chemical soil stabilization techniques which form a mixture of soil and binder and which are applicable in shallow layers, where expansion, rise of pressures and shear strength present a serious problem in geotechnical engineering. Expansion characteristics come to the fore at the surface, where soils are most susceptible to climate change. Numerous studies show that fly ash has a high level of effectiveness in reducing expansive characteristics of soils. When such soils were treated with fly ash, they

did not show a significant improvement of the current strength, but the seven-day strength was significantly higher than that of expanding soils. To prevail problems of soil expansion, unsatisfactory shear strength or stiffness, a cement or lime can be partially substituted with fly ash which has pozzolanic characteristics. When used for stabilization of foundation soil under roads, it is desirable to use fly ash of C class, according to ASTM D 5239 standard, because of its pozzolanic characteristics. F class fly ash is not recommended in this application, due to its insufficient pozzolanic characteristics, but instead it can be efficiently used in well graded sandy soil as suitable filler, where it can 'complement' the missing fraction of sandy soils. This would increase density, and thus compaction characteristics, and decrease permeability. As such it could be effectively used to form embankments.
Some tests, in which fly ash was used in grouting techniques, were carried out but the results are not as satisfactory as is the case when using it for the soil stabilization. However, good results were obtained in technique of deep soil mixing. This technique can achieve great depth of soil improvement, but the soil is improved only locally. This is in contrast with soil stabilization method, which encompass a large area but the depth is limited to a couple of meters of surface. Technology includes rotation of mixing equipment down to the planned depth of the column, Fig 2. On its way down, equipment breaks soil structure and, after reaching the final depth, the equipment is taken out to the surface by rotating in opposite direction than the one when going down. In its return, mixture containing binder is implemented in the soil, under pressure, thus forming the final cylindrical pillar of improved soil. In his research, Hansson [4] has completely substitute lime with fly ash in deep soil mixing technique. Results showed unsatisfactory results, considering that it was still necessary for mixture to have cement or lime. However, more satisfactory results were obtained as a result of mixing fly ash, cement and lime where fly ash involved with as much as 70%, indicating the potential use of larger amounts fly ash in the process of deep soil mixing.

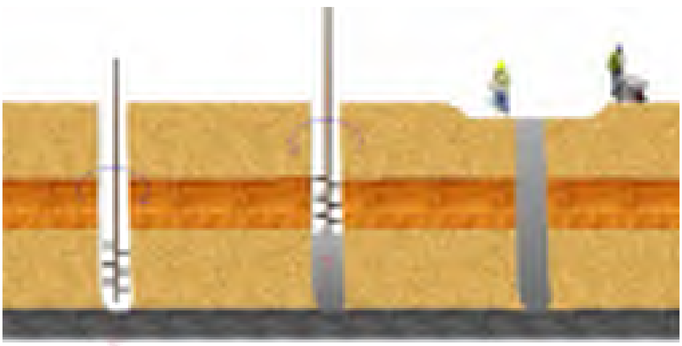

Figure 2 Deep soil mixing technique

A similar mixture, with large amounts of fly ash, was used by Ohishi [5]. He conducted a series of analysis and experiments in order to determine the characteristics of the mixture. The motive for research was the idea that this kind of mixture can be used in deep soil mixing for the execution of the foundation pit in the following way. The excavation of the pit included the construction of the diaphragm to greater depths since the subsoil had extremely unfavorable characteristics. By using deep mixing technique, an artificial layer within the thick layer of soft soil was formed and the diaphragm was ending in this layer. Requests for this layer implied sufficient strength to support the walls of the diaphragm, but at the same time it had to be soft enough so that diaphragm can be easily constructed.
The main problem that arises when using fly ash is finding a market in which there is a demand for it. The largest producers of coal combustion by-products (CCP) are associated in the organizations which represent the interests of CCP producers in the market. Furthermore, the

organizations of individual countries or regions are united in a worldwide umbrella organization of The Worldwide Coal Combustion Product Council. And while the annual production of fly ash quantities are rising, regulations and laws for their disposal all the more restrictive. However, governments of some countries provide significant incentives to civil engineering companies and to specific projects in order to encourage fly ash usage.

3 Sustainable ground improvement using slag

Slag is a waste product generated by the purification of metals, its casting and alloying and it can be divided into iron slag (blast furnace slag) and steel slag (steelmaking slag). Three types of blast furnace slag can be distinguished – air cooled, pelletized and granulated slag. Air-cooled slag is formed by slow cooling of slag deposited outside of power plants. The most common use of this slag is as aggregate in concrete and asphalt mixtures, as well as ballast material in railways or filling material in the embankments. Pelletized slag is obtained by cooling the molten mass with controlled amount of water, air or foam. This type of slag, due to low weight, is frequently used for construction of embankments on low bearing soils. Granulated slag is formed by rapid cooling the slag to the glassy state, using jets of water or air. As a product, sand-size grains are formed which, when grinded and with water added, show pozzolanic properties. This makes granulated slag usable for the cement industry and consequently in shallow soil stabilization and deep soil mixing technique. One of advantages of using granulated slag as cement components is in more efficient grinding than standard cement which saves considerable amounts of energy (only 10% of the total energy required for the production of Portland cement).
However, if the slag is coming from steelmaking, due to significantly lower quantities of C3S than in Portland cement, it does not have very good pozzolanic characteristics. It can be therefore potentially used as aggregate in concrete or as filling material in embankments, a railway ballast material or to form a road substructure. In studies, in which the comparison is made between layers containing steelmaking slag and layers with natural aggregate, it is shown that the slag layers achieve greater strength immediately after compaction of the material. Increased strength can be justified by the shape of the slag grain, which favors the "fixing" of grain after installation and compaction, forming a very solid, compact and durable surface that can be submitted to heavy traffic loads. This characteristic of being able to be utilized as aggregate is of interest in one of soil improvement technique – vibro-stone columns. A method of vibro-stone columns consists of formation of columns in the ground by using heavy vibrator which displaces in-situ ground and compacts the imported material, Fig 3.

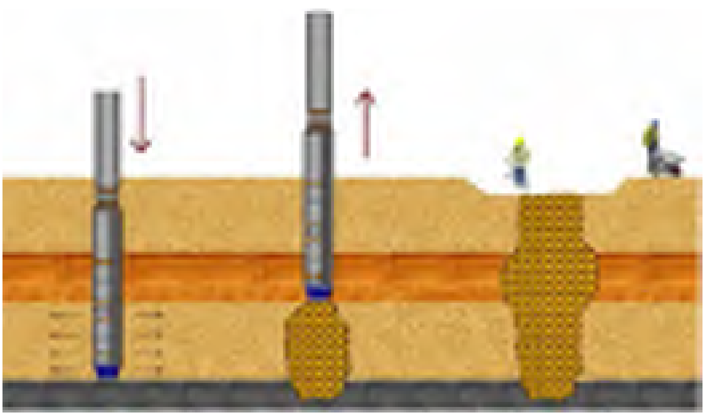

Figure 3 Vibro-stone columns (vibro–replacement) technique

This method has increasing trend of usage in Croatia. For example, on seven sections of Zagreb-Macelj motorway, 13 090 of vibro-stone columns were made, total of 104 780 m'. As aggregate filler, crushed stone obtained from quarries was used. This, in addition to the financial burden, also contributed to the depletion of natural resources. At the same time, Croatia is faced with increasing amounts of steelmaking slag to be deposited in landfills, what represent a major environmental problem.
An idea of using slag as an aggregate for vibro-stone columns is not new. Even in 'BRE Report 391 – Specifying vibro-stone columns' [6], a steelmaking slag is mentioned as one of aggregate potentially to be used in vibro-stone columns. In Saudi Arabia, until today, over one million tons of steelmaking slag was incorporated in soil through this technique [7]. However, this extent of application practice is not implemented worldwide, because possibility of application largely depends on type of slag and ground conditions. A range of testing procedures must be conducted prior to implementation. This includes both laboratory test of slag as well as field trial area where its interaction with surrounding soil will be examined. Extensive load trial tests provide most reliable interpretation of soil improvement, even though degree of improvement can be tested with alternative, simpler methods, such as Spectral Analysis of Surface Waves [8].
On Department of Materials, Faculty of CE in Zagreb, a series of tests were conducted tests on steelmaking slag from landfills in Split and Sisak [9]. This was done in order to examine the slag's potential as aggregate in concrete. In addition, some other properties of slag were presented, which authors considered as interesting to compare with standard dolomite aggregates. According to report [6], the material used as aggregate in the vibro-stone columns must be clean, hard and inert material, with sufficient hardness and weathering resistance to prevent degradation during installation which would lead to forming of sand or silt particles, and consequently lower friction angles. Material needs to have less than 5% of fine-graded fraction and grain size usually between 20 and 75 mm (depending on the method of execution). It also must suit the soil conditions in which columns are formed. British standards further require that the aggregate with value of the impact (BS 812-112) and the value of crushing (BS 812-110) larger than 30% are not suitable for use in vibro-stone columns, as well as aggregates which degrade or significantly weather when saturated. The following rules also apply: fragmentation resistance LA_{50} (ie Los Angeles coefficient <50), freeze / thaw resistance MS_{35}, no dicalcium silicate (Ca_2SiO_4) and iron disintegration, volume instability (expansion) less than 5% and sulphate content less than 0.5%. Table 1 presents some results of slag testing from Split and Sisak which are of interest for vibro-stone columns application.

Table 1 Possibility of using slag from Split and Sisak in vibro-stone columns

Criteria	Split (ST)	Sisak (SI)	stone column use ST / SI
clean, hard and inert	yes	yes	yes / yes
requested fraction	achievable	achievable	yes / yes
fine-graded fraction < 5 %	< 5 %	< 5 %	yes / yes
fragmentation resistance < LA_{50}	24.8 (LA_{25})	21.7 (LA_{25})	yes / yes
freeze / thaw resistance < MS_{35}	8.3 (MS_{18})	5.7 (MS_{18})	yes / yes
no Ca_2SiO_4 & iron disintegration	no	no	yes / yes
volume instability < 5 %	1.6 %	2.2 %	yes / yes
sulphate content < 0.5 %	< 0.5 %	< 0.5 %	yes / yes

Even though results from Table 1 show good potential for Croatian slag to be used in vibro-stone columns, additional test must be conducted prior to any finite conclusions. These especially include testing of value of the impact and value of crushing according to British standard. Some investigations made on slag from Germany [10] show that these parameters could be achieved since value of impact was in range of 18-22 % and value of crushing was in range 13-15%, all lower than requested 30%. Even though Steel Slag Coalition confirmed that usage of steel slag is safe for environment, extra care for heavy metals and other harmful substances in slag needs to be conducted. There are tests which have purpose of secretion testing, such as DEV-S4 method described in DIN 38 414. This technique was introduced in the third part of the EN 1744.

4 Fly ash and slag quantities in Croatia

In Croatia, the only coal combustion thermal power plant, Plomin, is located on the east coast of the Istrian peninsula. It is the only thermal power plant due to the fact that energy sources in Croatia are mostly focused on hydropower. Today two blocks of Plomin annually use about 800 000 tons of imported (USA, Colombia, Venezuela, Russia, South Africa, Indonesia and China) coal. When Plomin operates in normal mode, approximately 90 000 tons of fly ash is produced and it is used as raw material in cement factory. In present, there is no deposited fly ash in Croatia. However, until 2017 a construction of a new block is planned, with total annual coal usage increase up to 1.9 million tons. This would mean significantly higher amounts of produced fly ash. Same as in Croatia, where deposition of fly ash is managed properly due to small deposits, neighboring Slovenia has also 'settled problem' with fly ash. With total three thermal power plants in operation, Slovenia annually produces around 970 000 tons of fly ash of which most is used as fill material in construction and mining operations, while only a small portion is deposited. However, if fly ash is needed for soil improvement, it can be found in neighboring countries of Bosnia and Herzegovina and Serbia which still have huge deposits of fly ash due to large quantities of fly ash produced on annual basis. Additionally, these two countries still do not have a clear vision on managing fly ash and deposits are increasing. Four thermal power plants in Bosnia and Herzegovina, which is characterized by extremely large amounts of coal resources, produce annually around 2 million tons of industrial ashes (more than 9 million of coal is combusted). In Serbia, situation is even more alert, since five thermal power plants combust more than 35 million of coal per year, with close to 7 million tons of industrial by-products annually produced. Currently, only in Serbia there are around 180 million tons of deposited industrial by-products.
Unlike fly ash, Croatian slag deposits are huge. While some closed factories have exported their deposits of slag or used it to form embankments, 1.8 million tons of steelmaking slag from landfills in Sisak and 30 000 tons of steelmaking slag from landfills in Split are still on deposits without vision for their management. It is expected that amount of slag from Split will be doubled within one year. Slag from Sisak and Split is easily available as an aggregate for vibro-stone columns.

5 Conclusion

Soil improvement techniques lead to enhancement of soil characteristics, but require usage of great amount of natural resources and they can cause, to a lesser or greater extent, pollution of the surrounding environment. In the same time, urbanization leads to industrialization process which brings along a number of negative effects, such as deposition of large amounts of industrial waste materials. To achieve sustainability, fly ash and slag can be incorporated in soil improvement techniques and thus achieving dual positive effect – reducing pollution and extensive usage of natural resources while in the same time reducing waste material deposit amounts. Fly ash can be efficiently used as partial replacement for cement and lime in techniques of shallow soil stabilization or deep soil mixing. Depending of their origin and on type of their cooling, slags can be used also as replacement of cement or lime (granulated slag) or they can be used as an aggregate. This later application is especially interesting to be used as material in vibro-stone columns technique where it could replace natural gravel or crushed stone. In region, Bosnia and Herzegovina and Serbia have huge deposits of fly ash which can be potentially used for soil improvement. Unlike fly ash, slag deposits in Croatia are large and there is huge potential of using them as material for vibro-stone columns. Some preliminary laboratory results show that Croatian slag meets requirements for being used in vibro-stone columns, but more extensive laboratory and field test are necessary.

References

[1] The United Nations Human Settlements Programme, www.unhabitat.org, 21.01.2014.

[2] Cerić, A., Marčić, D. & Kovačević, M.S.: Applying the analytic network process for risk assessment in sustainable ground improvement, Građevinar, 65 (10), pp. 919-929, 2013.

[3] Kovačević, M. S., Simović, R., Bjegović, D., Rosković, R. & Peček, N.: Soil improvement with nano waste particles, Proceedings of the International Symposium: Non-Traditional Cement & Concrete III, Brno, pp. 362-371, 2008.

[4] Hansson, N.: Deep soil stabilization with fly ash, MSc thesis, Department of Earth Sciences, Uppsala University, Sweden, 2008.

[5] Ohishi, K.: Application of coal fly ash for improving soils prior to excavation, Proc. International Conf. on Engineering Materials, vol. 2, Ottawa pp. 531-541, 1997.

[6] Building Research Establishment Report 391 – Specifying Vibro Stone Columns, 2000.

[7] Slag product applications, www.globalslag.com, 01.02.2013.

[8] Kovačević, M. S., Marčić, D. & Gazdek, M.: Application of geophysical investigations in underground engineering, Tehnički vjesnik, 20 (6), pp. 1111-1117, 2013.

[9] Netinger, I., Jelčić Rukavina, M. & Bjegović D.: Mogućnost primjene domaće zgure kao agregat u betonu, Građevinar, 62 (1), pp. 35-43, 2010.

[10] Motz, H., Geiseler, J.: Products of steel slags an opportunity to save natural resources, Waste management, 21 (3), pp. 285-293, 2001.

METHODS OF SURVEYING IN ROCKFALL PROTECTION

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Abstract

Rockfalls are common in the countries with significant amount of mountain area. It is estimated that 27 percent of the world's land surface are mountains and about 12 percent of the world's population lives in the mountains. Due to that fact, there is a need of constructing new roads and railways along mountain slopes. In those areas, rockfalls can cause serious damage to infrastructure with possible human losses. Ensuring rockfall stability is a major safety goal along railways and highways. In order to simplify the complicated equations involved in rockfall modeling, computer simulation programs have been developed, both in two-dimensional and three-dimensional domains. The analysis of rockfalls includes estimation of trajectories of the boulders that falls from specific height along the slope. Once movement of the rock has been initiated, its falling behaviour is controlled by slope geometry, slope properties and boulder properties. The geometry of the slope represents one of the most important parameters and therefore it has to be accurately measured and determined as an input parameter for calculations of rockfall stability. Two-dimensional simulations require slope profile divided into several segments along slope, while the three-dimensional simulations require dense and accurate data in the form of digital terrain model. In this paper available methods for determining the geometry of slope are presented. Advantages and disadvantages of each method are discussed taking into account how it affects the final result of rockfall analysis.

Keywords: rockfall stability, photogrammetry, laser scanning, total station

1 Introduction

Croatia's transport infrastructure network plays significant role in the European network, as it is located at the crossroads of transit routes between western and south-eastern Europe and between the central Europe and the Mediterranean. The territory of the Republic of Croatia is crossed by three traffic corridors, Pan – European traffic corridors V, X and VII.
52 % of Croatian land surface, or almost whole coastal and mountainous regions of Croatia, is situated in karst. Karst is specific type of terrain mostly covered by barren limestone or dolomite, easily soluble rock characterized by strong karstification formation– progressive mechanical, physical and chemical disintegration of karstic rock, which leads to widening of joints system in rock mass. Together with weathering of exposed rock on the slopes, it can create systems of blocks, which can easily cause rockfalls under certain triggering situation. Triggers are usually weather conditions, in most cases intense precipitation or extreme temperature highs [1].
Croatian karst regions typically belong to medium scale surface karst (Dinaric region, Alps, Pyrenees, Appalachian Mountains, Australian Mountains, etc.), which stand out due to their thick (up to 8 km) carbonate Mesozoic and Paleozoic sediments with the pronounced tectonic fragmentation equally affecting the occurence of horizontal and vertical (speleological) formations. Croatian karst extends from Slovenia in the northwest to Montenegro in the southeast with its northern boundary running south from the city of Karlovac towards the east passing into Bosnia and Herzegovina [2].

The vast majority of sections of Croatian Highways A1, A6 and A7, in total 570 km of length, are situated in the hilly karst terrain with numerous slopes cut in the karstic rock. Even worse situation is on over 2200 km of National roads situated on the same terrain, where the slopes are steeper, older (and subsequently more weathered), closer to the road and less protected, owing to less strict demands for construction. In addition, many urbanized areas are situated directly under natural or man-made slopes, such as Opatija Riviera, city of Omiš and Makarska Riviera. In the last several years, a number of rockfalls was documented to have occurred in the mentioned highways, roads and urbanized areas [3].
Current approaches to the analysis of rockfalls require knowledge of the geometrical, structural mechanical properties of slopes, as well as boulder properties. The mechanical properties of rock slope material and boulder properties can be derived from in-situ and laboratory tests, whereas the geometrical characteristics only come from the field measurements.
The inaccessibility of steep rock slopes does not allow direct measurement of slope surfaces and collecting structural properties by traditional methods. Breaking point in slope geometry data acquisition system have come in the past several years, when some new measuring methods and techniques have become more available such as laser scanning and digital photogrammetry.

2 Total-station measurements

Total-station enables to measure heights, distance and angles to provide accurate positioning data. Total-station measurement depends greatly on the skills of the operator in terms of both capturing a sufficient number of points and selecting suitable points to represent the surface of the rock or slope. It is usually used for the purpose of two-dimensional simulations of rockfalls, when the slope profile is divided into several segments along the observed route. Rockfall simulations are carried out for each segment. Protection measures against rockfalls are designed according to the most critical segment. A major disadvantage of this approach is that the lateral dispersion of rock boulders is not taken into account. It doesn't provide sufficient amount of information required for rock fall analysis and can sometimes lead to an improper solution. An example is given in Figure 1.

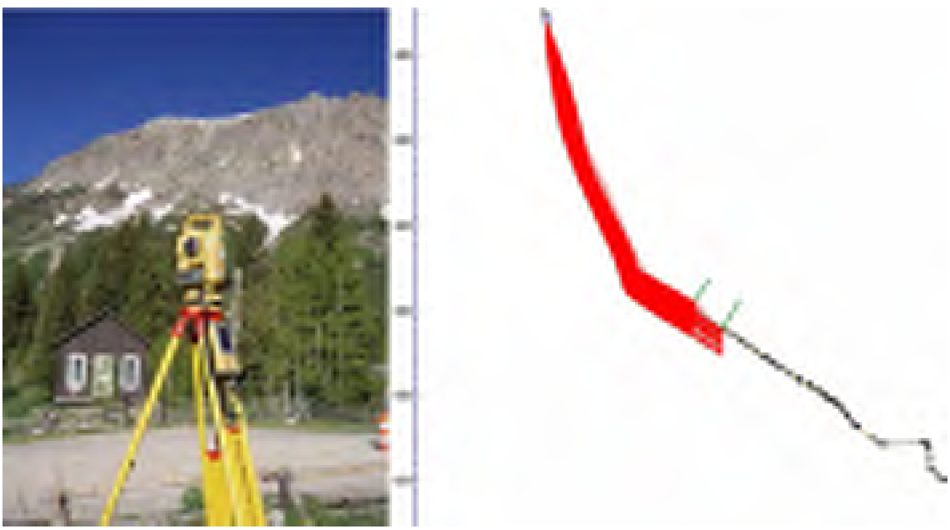

Figure 1 Typical total station equipment [5] on the left; Example of representative profile for rockfall analysis for the Stupica location on the state road D512 Makarska-Vrgorac [4] on the right

The rockfall protection barriers are calculated for the Stupica location on the state road D512 Makarska-Vrgorac in The Republic of Croatia. Stupica slope was divided into several representative profiles and two protection barriers were chosen according to the most critical slope [4]. Total station measurements are still usual method for obtaining geometrical characteristics of the rock slope for the rockfall stability analyses in Croatia. Structural properties of slopes and information about boulders' volumes are being collected by such traditional methods which can often take a lot of time.

3 Laser Scanning

Laser scanning or "3D laser scanning" is also known as ground based LiDAR which stands for "Light Detection and Ranging" system. LiDAR is an emerging three dimensional mapping technology that employs a laser and a rotating mirror or housing to rapidly scan and make image volumes of surficial areas such as: rock slopes and outcrops, buildings, bridges and other natural and man-made objects [6]. First it appeared in 1990's and to these days is becoming more and more popular in different fields.

There are two LiDAR types depending on the position of measuring equipment: – 1.terrestrial or ground based LiDAR (TLS), which refers to the tripod-based measurements and 2.airborne LiDAR (ALS), which refers to measurements made in the air by using a helicopter or an airplane. 3D laser scanners work by emitting immense number of laser beams to chosen surface and recording the reflection of the beam in order to accurately determine the distance to the reflected object (Figure 2).

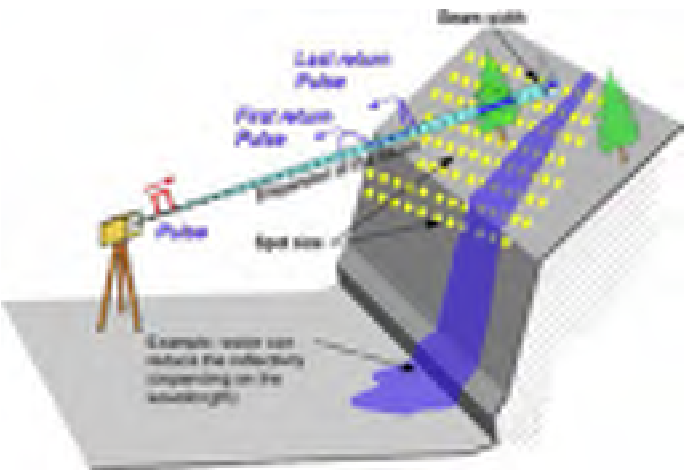

Figure 2 Principles of laser scanning for terrestrial lasser scanner (TLS) [7]

In this way 3D point cloud is created which represents an image of the rock slope face. The rotating mirrors or rotation of the housing allows millions of measurements to be made in just a few minutes which depend on the type of the scanner. Immediately after one pulse is received and measured, the scanner transmits another optical pulse slightly horizontal (or vertical – depending on the scanner) to the previous pulse using a rotating mirror or rotation of the housing. This process is repeated thousands of times per second, thus generating distance values for millions of points on a reflected slope. From the distance and the orientation of the laser pulse, the xyz coordinates associated with each reflected pulse can be determined. In addition, the intensity of the returned pulse is determined. In general, light coloured objects and closer objects give a higher reflection compared with darker objects and objects farther away. Together, the xyz coordinates and associated intensity values for millions of data points outputted by the laser make up the “point cloud” [6, 8]. Point cloud is the basic output from a 3D laser scanner. The most generic point cloud file format is a 3D coordinate file. The point clouds are then processed to extract geotechnical information, which includes discontinuity orientation, length, spacing, roughness, and block size. The first step in point cloud processing is to orient the point cloud into the real world coordinate system, based on data taken in the field. Most of the laser scanners are also equipped with high-resolution cameras, thus providing digital images of the scanned slope which can be draped onto the point cloud to provide a 3D colour digital terrain model, DTM. An example of a DTM from LiDAR measurements is shown in Figure 2. The main advantages of this technology are high accuracy, high resolution (mm to cm order) and very high data acquisition speed [8].

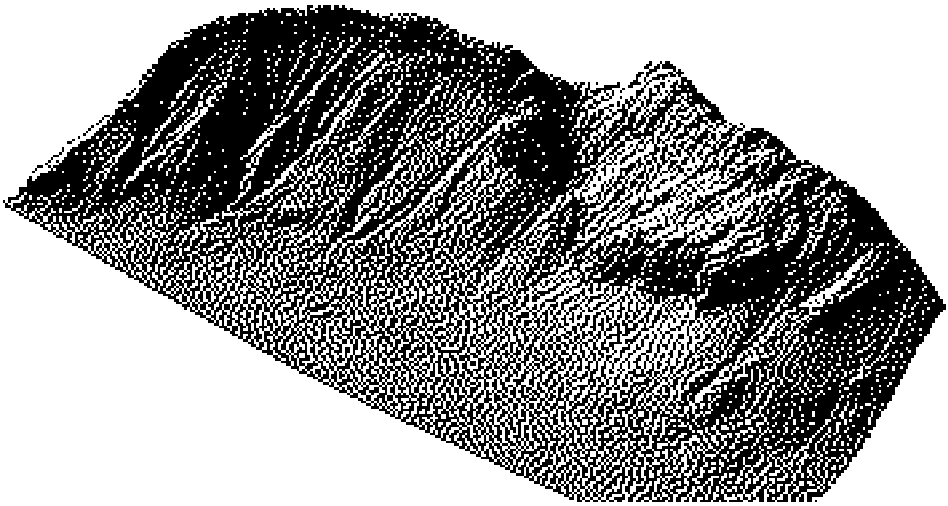

Figure 3 Digital terrain model (DTM) based on laser scanning, Veyrier-du-Lac, France [9]

According to Jaboyedoff [7], the application of laser scaning in rockfall engineering can be divided into four types: detection and characterization of rockfalls, hazard assessment and susceptibility, rockfall modelling and rockfall monitoring. To detect and characterize rockfalls, precise maps of surface need to be collected. Airborne LiDAR (ALS) has shown as a good tool for precise mapping of a large area, but still it is an extremely costly technique. It enables doing structural analysis at regional scale without entering the dangerous zone beneath a slope in a much faster and safer way, compared to the traditional methods. It is also very useful for hazard assessment since it enables to determine input parameters for rockfall hazard rating systems, such as: joint orientation and boulder volumes. It enables to accurately estimate volumes and positions of unstable boulders as input parameter for any rockfalls trajectory simulation, which enables to determine the influence area. Also, by repetitive measurement of rock slope face, 3D images are compared to previous images with precise incremental movements to cover big area of slope face, and not just one block, can be obtained in due time, by usage of an available software.

4 Digital photogrammetry

Photogrammetry is an optical method which allows to determine the geometric properties of objects to metrically reconstruct them by means of measuring and interpreting photographic images, using Image-based Modelling (IBM) [10].

There are two types of photogrammetry, depending on the position of the measuring equipment – 1. terrestrial, which refers to terrestrial photogrammetry and it refers to measurements from a fixed terrestrial location and 2.aerial photogrammetry, which refers to measurement made from an aircraft and is usually oriented vertically to the ground. Figure 4 shows a basic principle of photogrammetry. The 3D coordinates of a slope are determined from digital images taken of the same slope from different directions. It is necessary to collect at least two images of a slope, since the 3D coordinates are determined from at least two corresponding rays which are defined by perspective centre and each image point, showing spatial direction to the corresponding object point [9]. The advantage compared to LiDAR is that it directly provides colour image which enables to create textured 3D model.

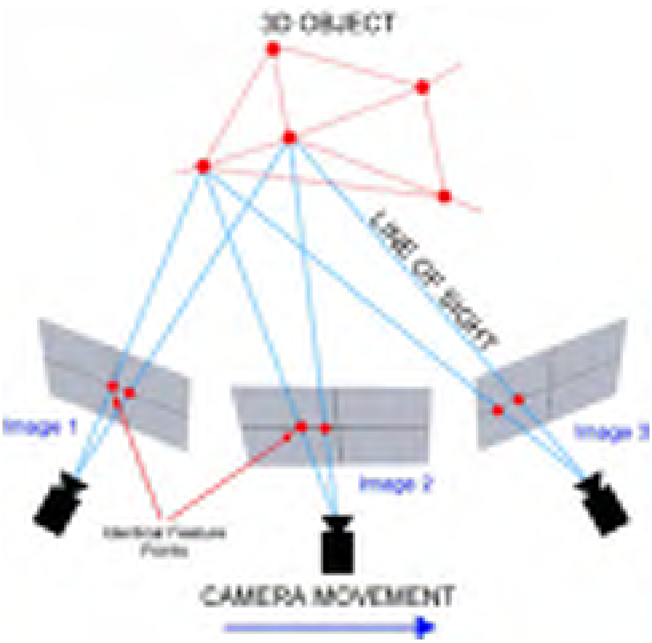

Figure 4 Basic principle of photogrammetry [11]

The equipment includes high resolution digital cameras, a tripod and markers which need to be attached to the slope as reference points. It is relatively inexpensive equipment, easy to handle which enables to take a large number of photographs in a short time. In terms of cost, photogrammetry equipment is less expensive than LiDAR, but photogrammetry softwares can sometimes be very expensive depending on data that need to be collected by such method. Today, there is a large number of photogrammetry software which are designed for extracting structural properties of the slopes such as number of trays and orientations of discontinuities. Those parameters also are basic input parameter in most of the rockfall hazard rating systems. Also, by comparing images taken in different times, displacement and deformations can be detected.

It must be emphasized that the advantage of LiDAR, as opposed to photogrammetry, is in scanning a slope that has vegetation. LiDAR light can penetrate through small openings between the slope and its vegetation to provide information of the rock underneath. On the other hand, obtaining a good result of photogrammetry depends on the available natural light available behind vegetation as it is affected by changes in light in different directions due to basic principle of photogrammetry to take multiple images of the same scene from different locations.

5 Conclusion

The traditional and new techniques in rock slope geometry data acquisition methods are described in previous chapters. Traditional total station measurements are very widespread in Croatia, though it doesn't provide sufficient amount of information to do for rock slope stability and rockfall analyses. In terms of cost, it still represents the lowest cost, but with major limitations. In the recent years, laser scanning and photogrammetry has become more and more popular in rockfall and rock slope stability applications in the whole Europe starting from Italy to France and Spain. There are different types of applications for each technique, from rockfall identification, modelling and monitoring. Even though both techniques are based on different principles, both produce a high-resolution 3D image of the analysed area. Obtaining the same amount of informations using total station would be highly impractical.
In terms of accuracy, compared to traditional method, both LiDAR and photogrammetry are suitable for rockfall stability analyses. It's believed that both of these methods provide accurate digital terrain models needed for the numerical analyses, as well as enough and accurate data with sufficient amount of information to do structural analyse for the rockfall hazard rating systems.

References

[1] Krautblatter, M. & Moser, M.: A nonlinear model coupling rockfall and rainfall intensity based on a four year measurement in a high Alpine rock wall (Reintal, German Alps), Nat. Hazards Earth Syst. Sci., 9, 1425–1432, 2009.

[2] Kovačević, M.S., Jurić-Kaćunić, D. & Simović, R.: Određivanje modula deformacije karbonatnih stijena u hrvatskom kršu, Građevinar, 63, pp. 35-41, 2011.

[3] Kovačević, M.S., Mirčeta, A. & Librić, L.: Numeričko modeliranje stijenskih odrona, Graditeljstvo-poluga razvoja, Lakušić, Stjepan (ur.), Zagreb, 79-88, 2012.

[4] Kovačević, M.S., Mirčeta, A. & Librić, L.: Tunnel Stupica – rockfall protection, Road and Rail Infrastructure II, Proceedings of Conference CETRA 2012, 7–9 May 2012, Dubrovnik, pp. 789-796.

[5] GPS software, http://rmbl.info/gis/gpssoftware.html, 23.2.2014.

[6] Kemeny, J., Turner, K.: Ground based LiDAR, Rock Slope Mapping and Assessment, Federal Highway Administration, Central Federal Lands Highway Division, Final Report, 2008.

[7] Jaboyedoff, M., Oppikofer, T., Abellan, A., Derron, M., Loye, A., Metzger, R., Pedrazzini, A.: Use of LIDAR in landslide investigations: a review, Nat Hazards, 61, pp. 5–28, 2012.

[8] Review of Techniques for Landslide Detection, Fast Characterization, Rapid Mapping and Long-Term Monitoring, Deliverable 4.1 of Safe Land, living with landslide risk in Europe: Assessment, effects of global change, and risk management strategies, /th Framework Programme, Cooperation Theme 6 Environment (including climate change) Sub-Activity 6.1.3. Natural hazards, 2012.

[9] Monnet, J.M., Clouet, N., Bourrier, F. & Berger, F.: Using geomatics and airborne laser scanning for rockfall risk zoning: A case study in the French Alps, The 2010 Canadian Geomatics Conference and Symposium of Commission I (ISPRS), Calgary, Alberta , Canada, 2010

[10] Remondino, F., El-Hakim, S.: Image-based 3D modelling-a review: Photogrammetric Record, pp. 269-291, 2006.

[11] Clemson University Restoration Institute, Warren Lasch Conservation Center, www.clemson.edu, 26.2.2014.

OPTIMIZATION OF GEOTECHNICAL INVESTIGATION WORKS DURING THE RECONSTRUCTION OF THE TRANSITION ZONES ON THE OLD RAILWAY LINES

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Abstract

This paper will analyse the optimal number of required data within geotechnical investigation works with the cost estimation of performed tests and the duration of the testing (line closure). Deviation of the geotechnical parameters obtained from various research works will be shown. Special attention will be paid to the review of the quality of non-destructive methods that require a minimum of line closures. In the paper a case study of extensive geotechnical, geophysical and laboratory investigation works will be presented at the location before and after the Buna Railway Bridge (railway line M104 Novska – Sisak – Zagreb) as part of an international research project SMART RAIL. Conducted investigation works consisted of: engineering geological mapping of the location, exploratory drilling with continuous coring, performance of standard penetration testing in boreholes, field vane testing in boreholes, excavation of trial pits, testing with nuclear densimeter, cone penetration testing with pore pressure measurement (CPTU), testing with flat dilatometer (DTM) and geophysical testing methods; geoelectric tomography, seismic refraction, MASW/REMI, downhole and crosshole seismic survey and ground penetrating radar (GPR).
The works are made with the aim of defining the physical and mechanical properties of the existing geotechnical structures (embankment) and foundation soil (primarily the stiffness of individual embankment layers), and general geotechnical conditions at the site. This is essential for the assessment and valorisation of any potential geotechnical problem that arise in the transition zones, and finally to create the reliable geotechnical models and to design the reconstruction of the new transition zone between the bridge abutments and geotechnical structures (embankment).

Keywords: transition zones, geotechnical investigation works

1 Introduction

Geotechnical investigations are necessary to define the soil layers and / or rock mass at each location, and geotechnical properties of these layers. From the results, the geotechnical model can be defined which shows geotechnical layers with related parameters.
Today is in use a lot of different geotechnical investigation techniques: extraction of core samples using a drilling machine, CPT tests, dilatometer tests, field vane tests and various geophysical methods.
Exploration drilling gives us continuous and direct view of materials, but the values of geotechnical parameters, obtained from the samples that were tested in the laboratory, are point data and represent a random sample.
CPT testing gives a continuous vertical display of soil with a series of parameters, but depends on the correlations with laboratory tests and depends on the interpretation of results.

Geophysical surveys provide 2D profile cross section at the location, but also depend on the correlations with the results of exploration drilling and parameters obtained by laboratory tests. In addition to the field and laboratory works, systematized and long-time collected data can be used. This, for example is, engineering geological, hydrogeological and seismic maps. These maps provide a general, outline of specific locations, and are done for each State. If data of previously conducted researches are available, they can be used for studies or geotechnical projects. Based on the background and current data necessary geotechnical investigation works can be reduced and rationalized.

2 Cost and cost-effectiveness of geotechnical investigation

When the costs and benefits of geotechnical investigation works are discussed there is a wide range of personal opinions "on this issue" among investors, designers as well as among geotechnical engineers. In most cases, these opinions are derived, formed and generalized into empirical opinions, based on personal contact with geotechnical issues in certain cases from practice. Depending on the "experience" (the number of individual cases and covered range of geotechnical problems), there are evaluations ranging from "unnecessary costs" to "extremely significant need for research". Neither of these extreme statements is applicable to all cases in practice, because there are always examples to the contrary. It can also be concluded that individual examples are generally not always adequate as arguments, because they contain a number of specific factors which generalize the conclusions.
Temple and Stukhart (1987) attempted, on the basis of data on dozens of cases in US practice, to assess the feasibility of geotechnical works by comparing the costs of inadequate investigations and additional costs during construction that they may have incurred, and the costs of additional investigations and savings in geotechnical solutions, [1].
Cases with available data showed that the savings are 2 to30 times higher than the total costs of geotechnical investigation works. These cases also indicate that the initial extensive and more complete investigation works can reduce or completely avoid the costs and time lag of subsequent studies.

3 Optimal conception of geotechnical investigation works

When selecting procedures and methods of investigation works, technological possibilities and disadvantages of different methods are taken into account, but also the costs involved in obtaining the required data reliability. Soil profile and geotechnical parameters are determined by:

- Drilling, extraction and testing of samples. This method is indispensable if we want to get verifiable material facts about the soil. Quality techniques of drilling and extracting samples are relatively expensive, and also the appropriate laboratory tests on undisturbed samples. The time required to obtain the data is relatively long.
- In-situ tests usually indirectly provide information about the mechanical properties of the soil, but can also be used to determine the soil profile. These tests provide useful results faster, avoiding the need for undisturbed samples and are generally cheaper than fully implemented direct examination. The lack of them is (relatively) less reliable, indirect use of the results and for example in static penetration tests (CPT), the absence of any soil sample from a specific depth profile.
- Surface geophysical survey methods are generally the cheapest, but to obtain detailed data of soil relatively unreliable. Surface methods can quickly cover a larger area, but usually give average results, and sensitivity to changes in soil materials is usually small. Better performance is achieved with larger, more clearly differentiated environment in combination with boreholes that provide a landmark for interpretation.

The optimal strategy of investigation works seeks to use most of the advantages of the previously mentioned groups of procedures for obtaining a larger number of high-quality data with maximum rationalization of time and money. Of course, any optimization makes sense to implement in cases where the massive investigations are expected or very high demands on the reliability of the results. In cases where there is limited field research, rationalization is achieved by a more detailed processing of the soil data, using the appropriate correlation and analysis. Using the benefits of various methods of investigation works can be carried out in the following way:

- On characteristic places on site, detailed investigation works with drilling, extraction of samples and detailed, rigorously conducted laboratory tests are carried out. Close to boreholes (or in them) in-situ testing is carried out, enabling the establishment of the local correlation between the two types of tests. These correlations are limited to a specific location and specific technological procedures and are not related with potential unreliability of correlations from the literature (although those can be used in the interpretation for comparison)
- For further application in the wider area only in-situ testing are used. If with a representative surveys the extremes at the site are covered, the in-situ tests are usually the interpolation within extremes. Any deviations or significant deviations from established local trends allow rational positioning of detailed research.

4 Transition zones

Transition zones are defined as parts of the railway track where a change of basic characteristics that define a railway structure in its entirety takes place, [2, 3, 4]. Under the basic characteristics following parameters are considered: substructure and superstructure stiffness, deformation of each substructure layer and each superstructure part, overall value of track deformations, geometric restraints. The transition zones in general represent the appearance of discontinuity in the track structure, [5]. Within the SMART RAIL project, focus is on solving the problem in the transition zones between two different types of substructure, between the open track on embankment and the bridge, [6].

4.1 Negative mechanisms that occur in the transition zones

Poor condition of the transition zones is a consequence of numerous complex and interrelated mechanisms. In order to find the best possible solution for solving the problems that happen in the transition zones, all the negative mechanisms which influence the behaviour of the track structure should be taken into account and analysed. Negative mechanisms that occur in the transition zones are:

- discontinuity in the stiffness of the track structure;
- differential settlements of the rail track structure;
- influence of rail services speed;
- influence of the direction of the train.

The above-mentioned degradation mechanisms, which in fact act each on their own, may also be conditioned by each other. Cyclic repetition of these processes accelerates degradation of track geometry, with reduced quality and safety of driving as an immediate consequence, [7].

4.2 Role of transition zones

A certain structural solution is in fact hidden behind the term “transition zones”. The main role of transition zones is to prevent sudden changes in stiffness of the load-bearing structural elements of the track. The aim is to minimize or prevent the occurrence of additional negative dynamic loads over a part of a transition zone, which additionally accelerate the track geometry degradation, with reduced quality and safety of driving as an immediate consequence. This can be achieved by linearly changing certain properties of the surrounding structures at a reasonable distance by dividing one differential change into smaller steps, i.e. dynamically irrelevant intervals, [2,4]. Ideally these inconsistencies occurring in parts of transition zones do not influence the performance of a passing train in terms of safety but rather they more often reflect upon the quality and comfort of rail services and other dynamic occurrences, [3].

5 Case study: transition zones at “Buna” bridge

“Buna” bridge, situated at the railways track M104 Novska – Sisak – Zagreb at km 398+422, is selected for the case study because of the obvious problems in the transition zones, immediately before and after the bridge. In the transitions zones irregularities in the geometry have been noticed, such as track unevenness and under ballast gaps, and vertical displacements of the whole track structure. These were caused by differential settlements and by dynamic impacts of the train due to the changes in the track stiffness. In order to detect the causes of degradation in the transition zones and also to perform the appropriate reconstruction of the same area it was necessary to collect all the information’s about the existing embankment and foundation soil.

Figure 1 “Buna” bridge

5.1 Performed geotechnical and geophysical investigation works

An extensive geotechnical and geophysical investigations have been carried out on site in March 2012. Field tests have been supplemented by numerous laboratory tests to provide accurate information about ground conditions. Based on investigation program, the following works were conducted at the "Buna" bridge site:

- engineering geological mapping of the site;
- drilling of four geotechnical structural boreholes (B1-B4) 12 m' in depth;
- excavation of four trial pits by the railroad line (R1-R4), and nuclear densimeter
- measurement with dynamic plate load testing of embankment bed;
- cone penetration testing with pore pressure measurement, CPTU on four positions (CPT1-CPT4);
- dilatometer testing on four positions near the CPTU locations (DMT1-DMT4);
- field vane testing, FVT, in four boreholes B1-B4;
- geoelectrical tomography. The surveyed profiles are designated with the initial and the final length of geotechnical profile and their designations are GT-1, GT-2, GT-3 and GT-4.
- seismic refraction surveying. The surveyed profiles are designated SRRF-1, SRRF-2, SERF-3 and SERF-4;
- seismic tomography, characteristic in-depth cross sections SRST-1 and SRST-2, recorded perpendicular to the railway track at the position of borehole B-2 and B-3, were determined on the basis of P-wave propagation velocities;
- MASW / REMI surveys. Results of MASW surveys are shown as 2-D seismic cross sections on both sides of Bridge location;
- boreholes seismic survey, downhole in four boreholes B1-B4 and crosshole between boreholes B1-B2 and B3-B4;
- ground penetrating radar, GPR, was used for profiling along railroad line;
- laboratory testing of soil samples.

Generally, ground on "Buna" bridge location is composed of fine grained surface material – high plasticity clay, in the middle part there is mixed material – silty sand, and the substrate consists of coarse grained material – poorly graded gravel. It can be said that the stiffness of the soil increases with depth. The increase in stiffness is best seen in downhole, crosshole and MASW-SRRF tests. One can see an increase of 100 MPa in clay up to about 600 MPa in gravel. Also the difference in stiffness is seen by the number of SPT strokes, which is in lower part of clay layer N=7-11 in sand N=5-7 and in gravel layer N=15-54.

CPT, DMT, FVT and laboratory tests are limited to the clay and sand layers. These layers are important for determining main part of deformation under railway line. According to CPTU and DMT tests moduli for clayey materials are 20-40 MPa, while dynamic moduli are 100-200 MPa. According to the overconsolidation ratio from CPTU test, which is OCR=5 – ‹10 for these soils, clays are mostly classified as very stiff fine grained soils. Values of geotechnical parameters are also shown graphically in Figure 2. Parameters obtained from laboratory tests, in situ testing and geophysical exploration are also shown. Material from depth 0,0 to 1,0 m, is railway embankment.

Generally it is very difficult to establish a link between non-destructive and destructive investigation works for the reason that the results of these methods are not mutually comparable. Non-destructive methods generally give us the dispersion of soil layers, and dynamic modules of deformability. What we have noticed is the relationship between the number of SPT strokes and Vs speeds. The ratio is in the coherent material ranged from 15-30 while in the incoherent material is between 5-20 what gives us the data for interesting correlations that should further be establish and verified in the future.

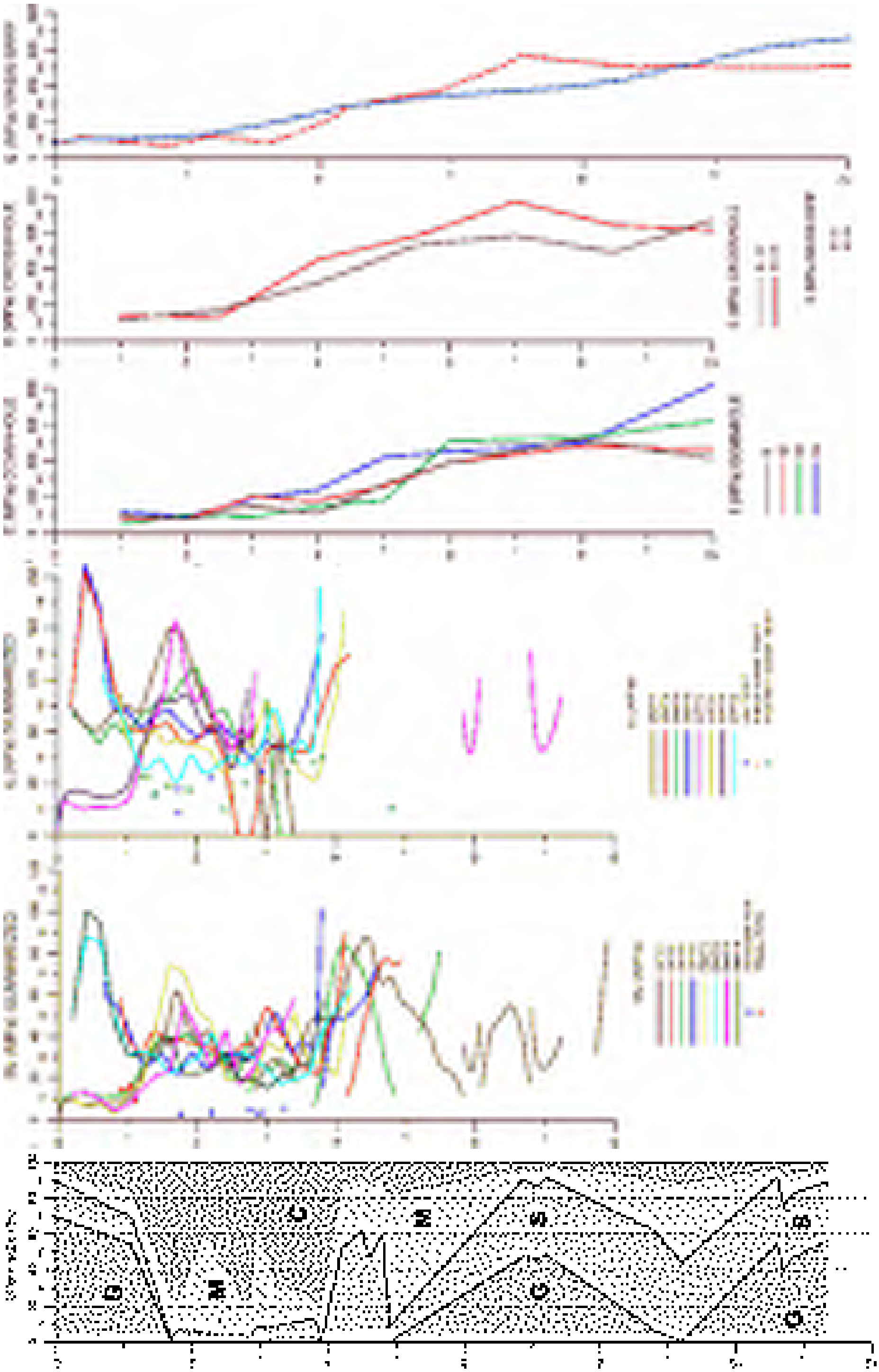

Figure 2 Obtained values of geotechnical parameters

6 Geotechnical investigation works optimization

Display of relationship between costs and applicability of certain geotechnical investigation works is shown in Table 1.

Table 1 Display of relationship between costs and applicability of certain geotechnical investigation works

Geotechnical investigation work	Note	Costs (ranged from 1-20, 1 the lowest cost - 20 the highest costs)	Applicability for the determination of shear strength (score ranging from 1-5, 1 weak applicability - 5 very good applicability)		Applicability for the determination of the stiffness (ratings in the range of 1-5; 1 poor applicability - 5 very good applicability)		Sum (points)
			Fine grained soil	Coarse grained soil	Fine grained soil	Coarse grained soil	
Drilling	The most reliable identification of layers and the mechanical characteristics of the soil	1	5	5	5	5	21
Trial pits	Limited to a small depth	14	3	4	1	3	26
Field vane test (FVT)	Conducted in the borehole	18	5				23
CPTU	Without any sample	16	1	5	1	4	30
DMT	Without any sample	16	4	4	4	5	36
Seismic refraction		17			2	1	[illegible]
Geoelectric tomography		17					[illegible]
Georadar		17					[illegible]
Downhole		18			5	1	20
Crosshole		15			5	1	20

Evaluation was carried out on the basis of cost and applicability of the work to determine the parameters of the soil. From the evaluation can be concluded that the CPTU and DMT tests have the best ratio of costs and quality of the data obtained. All these methods have limitations and conditions conquer. Therefore, it is impossible to compare with each other. Each project and each site is specific and therefore the most important factor is the human. In other words, for good optimization of each individual situation, the most important is the experience of the person who is caring out the optimization.

7 Conclusion

Optimization of investigation works must achieve the following:
- optimum relationship between the quality of the data obtained and the cost of investigation works;
- if possible, carry out investigation works without closing traffic on the railroad, then again the criterion of cost reduction is present;
- in the framework of the conducted works it would be necessary to systematically monitor the investigation works on the railways, including the creation of database and continuous settlement monitoring of transitional zone.

The results of this study suggest the following:
- reduce the scope of investigation drilling and trial pits at the minimum allowed;
- conduct an analysis of all previous investigation works on railway lines;
- use to the fullest extent CPTU tests and geophysical profiling

Based on the analysis of the obtained results during the rehabilitation of the existing transition zones, the authors of this article recommend the following investigation works as optimal: drilling with laboratory testing and depending on the type of foundation soil and geotechnical structure, performing CPTU or DMT method of soil profiling and geophysical methods: geoelectric tomography or seismic refraction.

Acknowledgements

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References

[1] Temple, H.W.B., Stukhart, G.: Cost effectiveness of geotechnical investigations, Journal of Management in Engineering, ASCE, Vol.3, No. 1, p.8-19, 1987.

[2] Esveld, C.: Modern Railway Track, Second Edition, TU Delft, 2001.

[3] Rossmann: RHEDA 2000® Transitions, Rail One, Brno, 2009.

[4] Jenks, C.W.: Reserch Results Digest 79 – design of track transitions, Washington, 2006.

[5] Vertical Elasticity of Ballastless Track, UIC, Paris, 2008

[6] Smart Maintenance and Analysis of Transport Infrastructure, Collaborative project (SMART RAIL), Description of work, FP 7 project number: 285683, Dublin, 2011.

[7] Vajdić, M., Oslaković, I., Kralj, S.: Transition zones on the railway track – overview, 2nd International Conference on Road and Rail Infrastructure – CETRA 2012, 7–9 May 2012, Dubrovnik, pp. 633-640.

[8] Tomislav, I.: Racionalizacija geotehničkih istražnih radova na terenu i laboratoriju s primjenom rezultata u projektiranju temelja objekata, Zagreb, December 1991.

[9] Program of geotechnical investigations works for the “Buna” bridge on the railway line: Zagreb-Sisak-Novska in Croatia, Institut IGH, Zagreb, January 2012.

[10] Ground investigation report for the “Buna” bridge on the railway line: Zagreb-Sisak-Novska in Croatia, Institut IGH, Zagreb, April 2012.

INFLUENCE OF LAYERED GEOSYNTHETICS ON CBR OF CLAYEY SUBGRADE WITH SOIL-GEOSYNTHETIC INTERACTION

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Abstract

The present research focuses on improvement of various soil properties and strength parameters of clayey subgrade using geogrid (biaxial) and geotextile (nonwoven) placed through various reinforcement systems viz. Single layer reinforcement system (SIL), composite layer reinforcement system (COL), multi layer reinforcement system(ML) at various position/levels of subgrade thickness. A high strength geotextile and a biaxial geogrid are used as geosynthetic reinforcement materials in this study. Soaked and Unsoaked CBR tests were performed on expansive soil (CI and CH soil) using above mentioned reinforcement systems. The main objective of this study is to know the most efficient geosynthetic reinforcement system and position of reinforcement for medium expansive soil (CI) and highly expansive soil (CH) of Gujarat region.. Given results depicts that infusion of geosynthetic reinforcement at various levels of subgrade note only improves CBR value but overall its increases the structural stability of the subgrade. Out of various subgrade systems composite reinforced system at position of h/5 gave the maximum CBR value and its calibration with modulus of subgrade reaction (K) was quite compatible.

Keywords: subgrade, geosynthetics, reinforcements, expansive soils, CBR value

1 Introduction

Pavement is a durable surface having materials laid down on an area subjected to sustain mainly the vehicular traffic, such as a road or highway. Subgrade is the foundation layer; the structure must eventually support all the loads which come on to the pavement. The performance of the pavement is affected by the characteristics of the sub-grade. Desirable properties which the sub-grade should possess are: strength, ease of compaction, permanency of compaction and permanency of strength, low susceptibility to volume changes and frost action. Since sub-grade soils vary considerably, the inter-relationship of texture, density, moisture content and strength of sub-grade materials includes maximum dry density (MDD), optimum moisture content (OMC), CBR and E-value of sub-grade material. To understand the behavior of failure and minimize it, a compressive laboratory program was made to study the strength characteristics of both reinforced and un-reinforced soil. This work describes the beneficial effects of reinforcing the sub-grade layer made up of expansive soil (CH & CI) with either single, double or multiple alternate layers of geo-grid and geotextile at different positions to determine the optimum position based on CBR test values.

1.1 Theoretical developments

The concept of reinforcement is not new. Early civilizations commonly used sun-dried soil bricks as a building material. And in their experience it became an accepted practice to mix the soil with some kind of fiber to improve the properties (Dean, 1986)[1]. The materials used as reinforcement in sub-grade soil can vary greatly, either in form (strips, sheets, grids, bars, or fibers), texture (rough or smooth), and relative stiffness. Nejad and Small (1966)[2] investigated that geogrid could significantly decrease the permanent deformation in the pavement by 40% to 70%. Ling and Liu (2001)[3] carried out some static and dynamic tests on model sections to find out the contributions of geosynthetic reinforcement to the stiffness and strength of asphalt pavements. The study showed that the settlement over the loaded area of reinforced pavement as reduced when compared with un-reinforced pavement. Also C.R. Lawson (1995)[4], Asha (2010)[5], M.S. Natraj (1997)[6], Minu Michal (2009)[7], they all studied about geosynthetic as reinforcement.

1.2 Laboratory investigations

Soil collected from Kheda region and Sanand Region of Gujarat State has been used as subgrade material in the experiments. The soil is classified as CI and CH. Index properties are found out in the laboratory. The properties of the soil used in the study are given in Table 1. Geosynthetic material was carried from TECHFAB INDIA and also property of goetextile and geogrid as in Table 2 and Table 3 geogrid and geotextile respectively.

1.3 Testing methodology

California Bearing Ratio tests were conducted on CI and CH soils, unreinforced and reinforced with a single layer and double layers of geotextiles or geogrids and either composite layer of geogrids & geotextile.To reinforced a sample, composite material of geogrids & geotextile were placed in a single layer at different positions: h/2, h/3 & h/5 & multi layer of geogrids & geotextile were placed at h/3 & h/6 of the specimen height from the top surface. The dry weight required for filling the mould was calculated based upon the maximum dry density(MDD) and corresponding optimum moisture content was achieve from standard proctor test. A total of 15 samples of reinforced and unreinforced both unsoaked were tested (as per IS 2720-16)[4] the load penetration readings were noted for all the samples to obtain CBR value, modulus of subgrade reaction (K) and secant modulus.

Table 1 Physical properties and classification

Index Properties	Kheda Region	Sanand Region
Liquid Limit (L_L)	35.75%	66.30%
Plastic Limit (P_L)	19.09%	23%
Soil Classification	CI	CH
Optimum Moisture Content (OMC)	21.70%	22.55%
Max. Dry Density (MDD)	1.60 gm/cc	1.645 gm/cc
Specific Gravity	2.59	2.60

Table 2 Properties of Geogrid

Sr. No	Experiment Work		ASTM Code	Biaxial geogrid
1	Ultimate tensile strength (KN/m)	MD	D-6637	90
		CD		90
2	Elongation at maximum load (%)	MD		15
		CD	D-6637	15
3	Tensile strength at 2% elongation (KN/m)	MD		11
		CD		9
4	Tensile strength at 5% elongation (KN/m)	MD	D-6637	21
		CD		
5	Aperture Size (±2mm)	MD X CD		23x23

Table 3 Properties of Geotextiles

Sr. No	Experiment work	ASTM Code	Unit	Nonwoven geotextiles
1	Mass per unit area	D-5261	g/m^2	200
2	Grab Tensile strength	D-4632	N	720
3	Elongation @ break	D-4632	%	60
4	Trapezoidal Tear	D-4533	N	300
5	Puncture Strength	D-4833	N	400
6	Permeability/ Flow rate	D-4491	$l/m^2/s$	100
7	Mullet Burst	D3786	kPa	2175
8	Thickness	D-5199	mm	1.6
9	Apparent Opening Size	D-4751	µm	150

2 Plots for Comparison

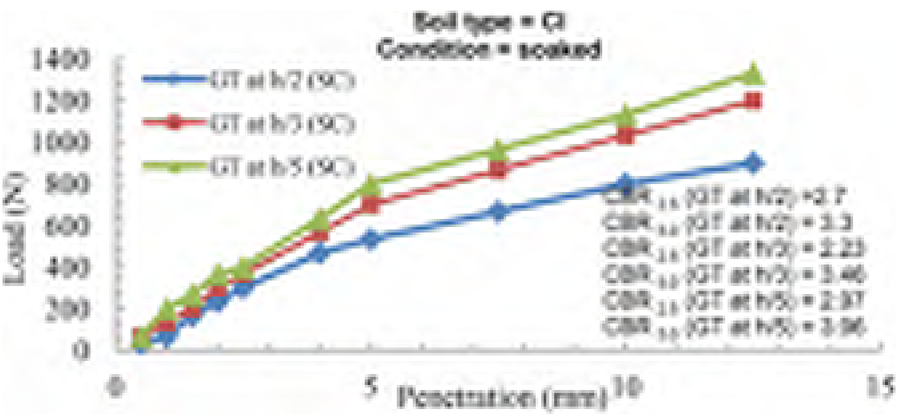

Figure 1 Load v/s penetration curve for SIL of GT at different position (SC)

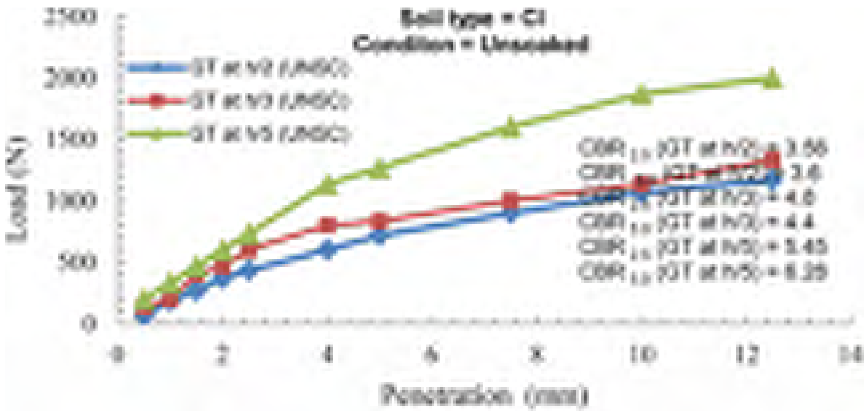

Figure 2 Load v/s penetration curve for SIL of GT at different position (UNSC)

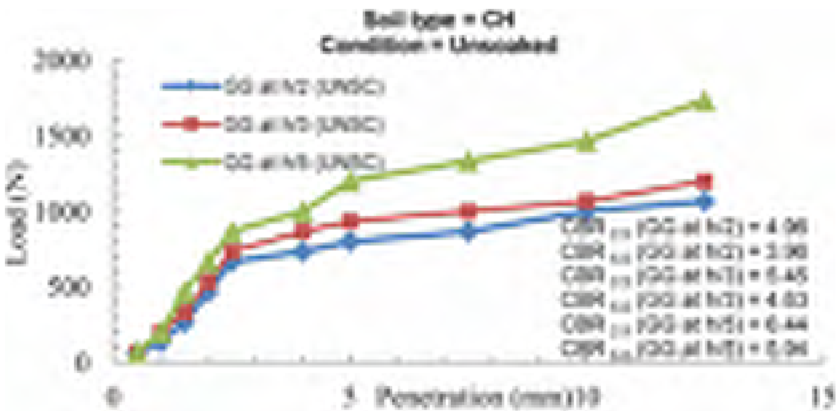

Figure 3 Load v/s penetration curve for SIL of GG at different position (UNSC)

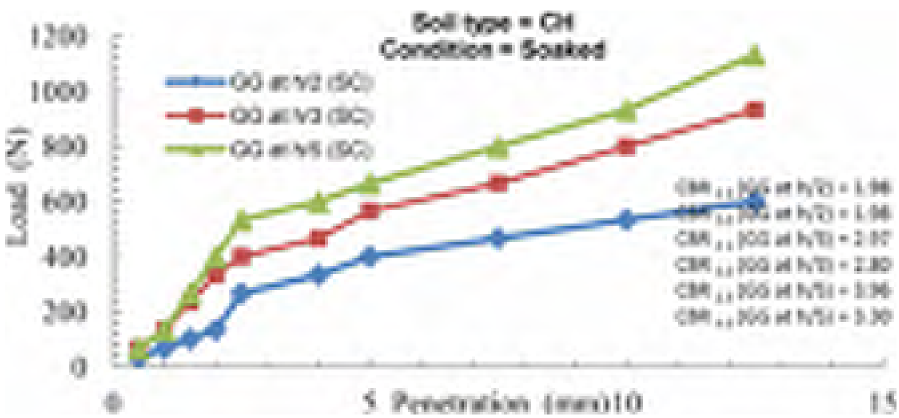

Figure 4 Load v/s penetration curve for SIL of GG at different position (SC)

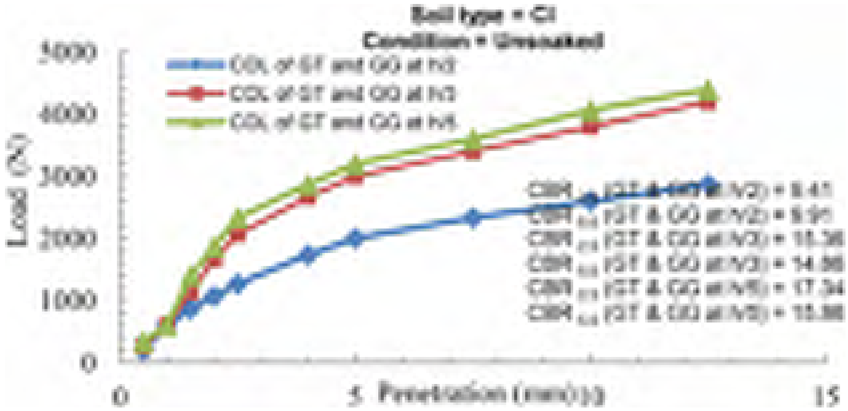

Figure 5 Load v/s penetration curve for COL for various positions of GG and GT

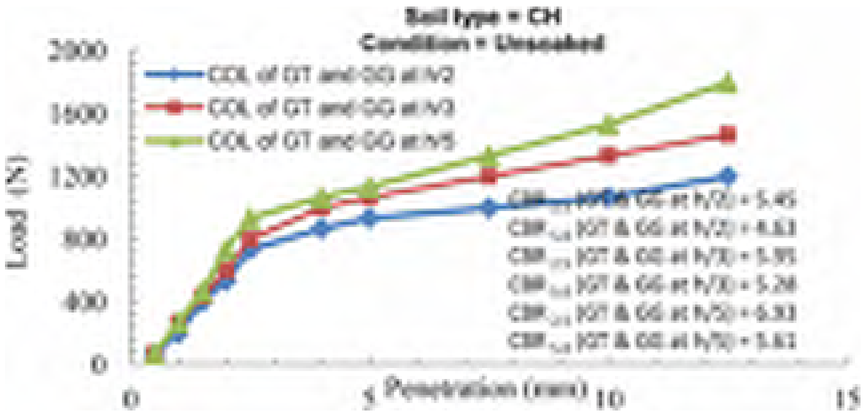

Figure 6 Load v/s penetration curve for COL for various positions of GG and GT

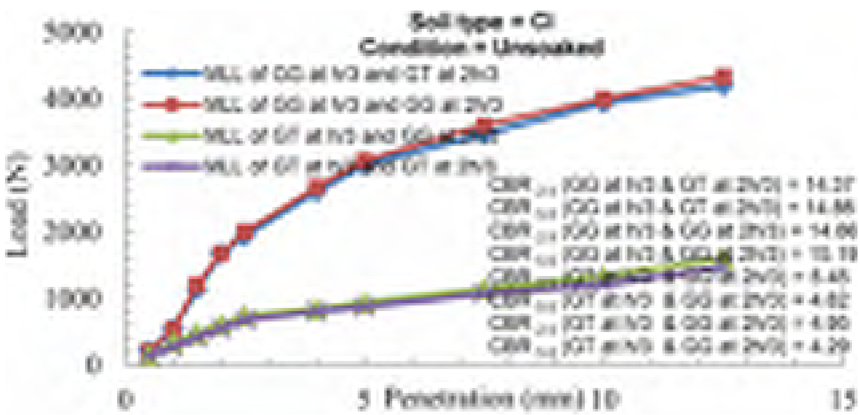

Figure 7 Load v/s penetration curve for MLL for various positions of GT and GG

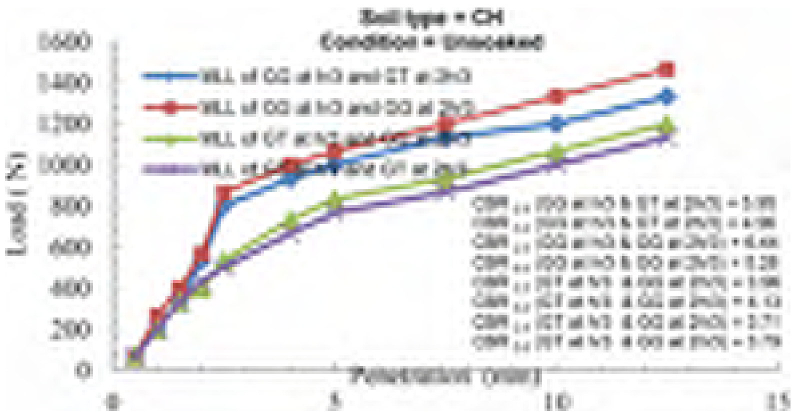

Figure 8 Load v/s penetrations for MLL for various positions of GT and GG

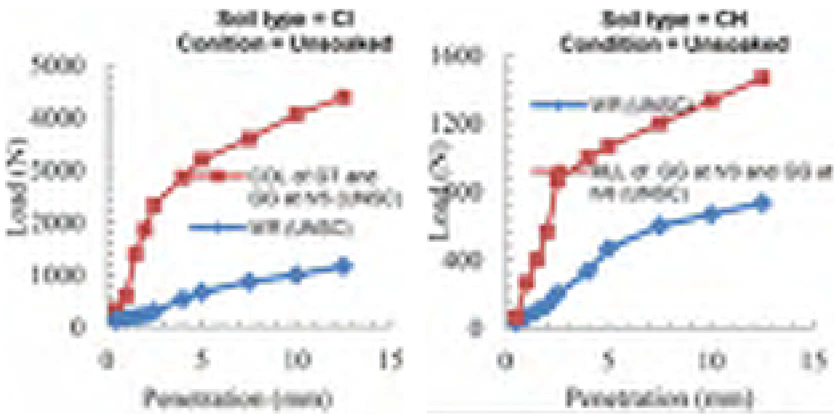

Figure 9 Load v/s penetration curve for comparison of WR with COL and WR with MLL for GT and GG at h/5 (UNSC)

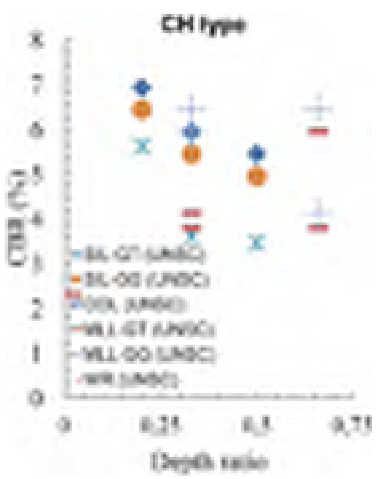

Figure 10 Comparison of CBR v/s depth ratios for WR, SIL, COL and MLL

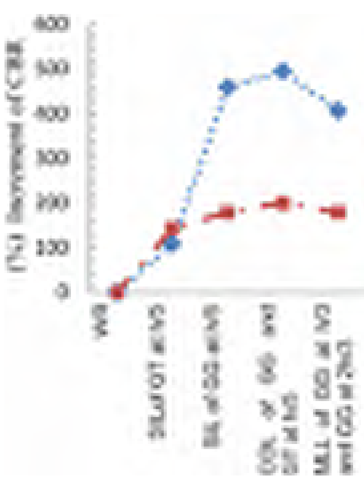

Figure 11 Percentage increment of CBR v/s various efficient reinforcement systems

3 Results, analysis and discussion

The relative performance of various geosynthetic reinforcement systems in terms of CBR value and its interaction with subgrade is discussed below.

3.1 Effect of single layer of geotextile (GT) and Geogrid (GG) at various locations

From figures 1, 2, 3, and 4 it is observed that geotextile placed at h/5 shows more load carrying capacity compare to h/3 and h/2 under soaked and unsoaked condition. From the plot it is reflected that load carrying capacity of geotextile at h/5 is nearly 1.1-1.5 times to h/3 and h/2 at same penetration of 5mm and 10mm under soaked condition and 1.5 – 1.8 times to h/3 and h/2 under unsoaked condition. In case of CI soil Geogrid placed at h/5 shows more load carrying capacity compare to h/3 and h/2 under soaked condition while under unsoaked condition geogrid is effective both at h/5 and h/3. From the plot it is reflected that load carrying capacity of geogrid at h/5 is nearly 2-2.5 times to h/3 and h/2 at same penetration of 5mm and 10mm under soaked condition while load carrying capacity remains almost same at h/5 and h/3 under unsoaked condition. While in case of CH soil geogrid placed at h/5 shows more load carrying capacity compare to h/3 and h/2 under soaked condition while under unsoaked condition geogrid is effective both at h/5 and h/3. From the plot it is reflected that load carrying capacity of geogrid at h/5 is nearly 1.1-1.7 times to h/3 and h/2 at same penetration of 5mm and 10mm under soaked condition while load carrying capacity remains almost same at h/5 and h/3 under unsoaked condition.

3.2 Effect of combined layer of geotextile (GT) and Geogrid (GG) at various locations

From Fig 5, and 6, it is observed that combined composite layer of geogrid and geotextile placed at h/5 shows more penetration compared to h/3 and h/2 under both soaked and unsoaked condition for both types of soil i.e. CI & CH. From the plot it is clear that the nature of curve is quite linear in case of CH soil when composite reinforcement placed at h/5, while some curvature is seen for h/3 and h/2 for CI type of soil. Curve of h/5 remains above curve of h/2 and h/3 both under initial and final penetration. From the plot it is also observed that load carrying capacity of combined reinforced mass at h/5 is nearly 1.25-1.7 times to h/3 and h/2 at same penetration of 5 mm and 10 mm under soaked and unsoaked condition. Resistance to penetration is observed for initial loads when composite reinforcement placed at h/5 indicating the superimposed layer of geotextile on geogrid offers more resistance because of its high interface friction, more compression capacity and high strain compatibility. More load carrying capacity with less penetration is observed in case of CH soil at initial level up to 5 mm but thereafter in increases linearly showing constant amount of penetration with increasing trend as compared to decreasing trend seen in CI soil.

3.3 Effect of multi-layered composite layer of geotextile (GT) and geogrid (GG) at various locations

From fig 7 & 8, it is observed that multi layer reinforcement (one layer of geogrid and one layer of geotextile) placed at h/3 shows more penetration compared to h/3 under unsoaked condition for both CI and Ch soils. Four combinations of multi layered system were tested using CI and CH soil in following way: GT at h/3 & GG at h/6, GG at h/3 & GT at h/6, GG at h/3 & GG at h/6, GT at h/3 & GT at h/6 respectively. Combination of GG at h/3 & GG at h/6 shows maximum load carrying capacity at 5mm and 10mm penetration compare to other while combination of GG at h/3 & GT at h/6 showed better results compared to others. It is also noted that initial nature of curve for all combinations is same except in case of GT at h/3 & GT at 2h/3 which is quite distinct showing very less value of load carrying capacity both at 5mm and 10mm penetration. Curves almost remain linear during initial period of load showing less penetration compare to geotextile which in other words can be said that geogrid offers more resistance to penetration under 5mm and 10mm. Curve of h/3 remains above curve of 2h/3 both under initial penetration and final penetration. Resistance to penetration is observed for initial loads when geogrid placed at h/3 and at 2h/3 indicating that biaxial geogrid placed near to surface offers more resistance because of its high tension capacity, more interlocking friction and rigid strain compatibility (more resistance to deformation). Also cross plane (lateral) resistance to load is much more compare to geotextile. Here also as we move away from surface (load area) effect of reinforcing soil decreases.

3.4 Effect of WR and various reinforcement systems under various conditions

Referring to figure 9 showing the comparison of load vs. Penetration plots between unreinforced soil (WR) and single layer reinforced soil (SIL) with geotextile (GT) at h/5 for CI and CH soil, we can conclude that for unsoaked condition load carrying capacity is more compared to soaked condition and with infusion of single layer geotextile load carrying capacity increases rapidly both at 2.5mm and 5mm penetration. For COL with geotextile (GT) and geogrid (GG) at h/5 for CI and CH soil the nature of plot is quite similar both in case of CI and CH soil i.e. very narrow increasing trend in load with more deformation. Multi layer reinforcement system is more beneficial as higher resistance is offered by reinforcement against shearing phenomena of CI soil particles as both layers of geogrid are offering tenacity, shear resistance and lateral spreading of soil.

3.5 Comparison of CBR for WR and various reinforcement systems under various conditions

Referring to figure 10 & 11, showing comparison of composite reinforcement and multi-layered system reinforcement it is very clear that multi-layered system reinforcement is more advantageous as compared to composite system. Hence it can be said that location of reinforcement is playing a vital role CBR value or we can say that CBR value is related to the location of reinforcement particularly in case of expansive soils. In case of composite reinforcement for CI & CH soil, location of reinforcement at h/5 shows maximum CBR value of 17.35% and 6.3 % respectively, while if we compare layered reinforcement system then combination of GG at h/3 & GG at h/6 shows increment of CBR value of 15.2% for CI soil and 6.44% for CH soil under unsoaked condition. Referring to fig 9 which shows increment of CBR value for composite and multi-layered system, it is very clear that composite system shows nearly 480% more CBR value for CI soil while 200% more CBR value for CH soil and in case of multi-layered system it shows 396 % for CI and 179% for CH soil respectively. This concludes that under unsoaked condition, composite geogrid shows better results and is more efficacious at location h/5. On comparing CBR results of reinforced soil using MLL of GG at h/3 & GG at 2h/3 and GT at h/3 & GG at 2h/3 separately the CBR value increases from 178% and 56% for CI and CH soil. Very peculiar phenomena was observed in case of CBR samples interfaced with geogrid, that as load increases from zero to 50kg penetration value is very less, just 2mm it means much of load is occupied as seating load and because of high rigidity of MLL, GG at h/3 & GG at 2h/3 of, almost interfaced soil layer would get compressed to same amount to occupy aperture size or geogrid superimposing with soil layer, while in case of geotextile at 50kg load penetration was almost nearly 44mm double to that of geogrid which indicated that as geotextile fiber is soft enough that compression of soil sample along with compression of geotextile is such that at interface, tension is developed between partile-to-partile along with bearing capacity failure at the end is noted. In case of geogrid sliding failure is observed at the end of test. Computing modulus of subgrade reaction (K) from CBR value it is observed that value are within acceptable limits and higher k value s observed for geogrid at h/5 as shown in table 4.

Table 4 Comparison of CBR Value and modulus of subgrade

Location		CBR [%]	K-value [kg/cm²]
GG @ h/5	Composite (CI)	17.35	6.51
GT @ h/5	Composite (CH)	6.93	4.81
GG @ h/3 & h/6	Multi-layered (CI)	15.2	6.23
GG @ h/3 & h/6	Multi-layered (CH)	6.44	4.64

4 Conclusions

The following major conclusions drawn from this study:
- Improvement of clayey subgrade by using any geosynthetic reinforcement system yields gain in CBR value as compared to unreinforced troublesome expansive soil.
- Reinforced subgrade soil system performs better than unreinforced in terms of CBR value increment and reduction in penetration at specific load.
- Composite reinforcement system shows better and reliable results as compared to multi-0layered system both for CI and CH soils. Location of geosynthetics at particular height of subgrade and its interaction with soil under various loads plays a vital role in predicting failure mechanism of subgrade.
- Highest CBR value is achieved with subgrade reinforced with geogrid at location h/5 for given OMC and densification. While in case of multi-layered system GG ath/6 and GG at h/3 showed maximum CBR value under unsoaked condition.
- Modulus of subgrade reaction (k) value is much nearer to standard range and can equally be applied for settlement predictions and design of pavements.
- Design of clayey subgrade using geosynthetic reinforced systems in terms of relative efficiency of various reinforcement systems at various positions of subgrade and their effect on the load-penetration behavior.

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References

[1] Frietag, D.R.: Soil randomly reinforced with fibers. Journal of geotechnical Engineering, ASCE, Volume 112, No. 8, pp. 823-826, 1986.

[2] Nejad, F.M. and Small, J.C.: Effect of geo-grid reinforcement in model track tests on pavements. Journal of transportation Engineering, ASCE, Volume 122 (6), pp. 468-474, 1996.

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FEM ANALYSIS WITH SPECIAL FOCUS ON SOIL-STRUCTURE INTERACTION OF FLOATING SLAB-TRACK INFRASTRUCTURE IN HIGH SPEED RAILWAY EMBANKMENTS

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Abstract

Use of Floating Slab Track (FST) type infrastructure systems in high speed railway (HSR) embankments is becoming increasingly popular in the world today as well as a mean of vibration isolation and safe and fast rail travel. The main emphasis of this study is on the application of non-ballasted concepts for high-speed operation used in the design of Far Eastern HSR embankments and a manufactured floating slab track system. In this paper, finite element method (FEM) is used to model soil-structure interaction. Effects of soil stiffness (k_s) are carefully investigated. Longitudinal settlements are obtained and checked against allowable values. The study has confirmed the quality and reliability of the FST systems, which continue to have huge use in high speed rail design-construction projects nowadays.

Keywords: soil-structure interaction, coefficient of subgrade reaction, High Speed Railway embankments, slab track

1 Introduction

New slab track designs are being developed in the world today, in order to come across with a need for safe and fast passenger-load carriage along heavy transportation service lines that will operate with low maintenance costs. The so-called slab track is a concrete or asphalt surface made of stiff and brittle materials with resilient components to provide the required elasticity. Factors like ground conditions, life cycle duration, cost, construction time, availability and durability are the main factors in designing railway lines nowadays. With regard to the specified factors, modern slab-tracks are replacing the traditional ballasted track designs nowadays. The significant increase and popularity is mainly due to the low maintenance and efficient life-cycle costs. However both ballasted and unballasted track designs have their advantages and disadvantages and in some cases still standard ballasted track designs may have more advocates, as they are widely used in high speed operation areas, especially when embankments are built on soft clays or soft peat layers, as well as in earthquakes areas. There are basically 2 types of embankment lateral-sections, namely The European type – ballasted and the Far-Eastern type – slab track (Fig. 1) though the longitudinal sections are very similar. The trend shows that; although the standard ballasted concepts are still popular in general, they will lose their attractiveness in favour of slab tracks systems, due to this new attitude. In this study we have analyzed the Far-Eastern case. In the Far-Eastern slab track type embankment one fill strata called 'Uncemented-Prepared Subgrade Layer (U-PSL)' is replaced with a cemented one called 'Cemented-Prepared Subgrade Level' (C-PSL) (Fig. 1a) [1].

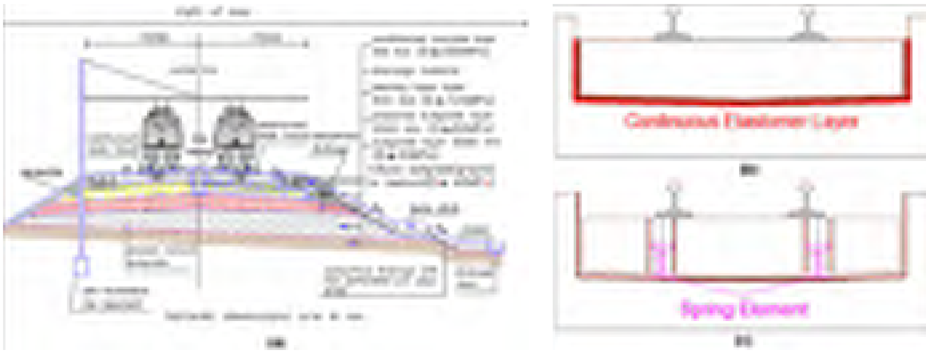

Figure 1 (a) Cross-section of a HSR embankment (Far-Eastern type) [1], (b) FST Type 1, (c) FST Type 2

Moreover, vibration isolation and sound effects' reduction play an increasingly important role during design and performance evaluation of a HSTR. With the increase in traffic intensity, it becomes more and more difficult to ensure comfort, security and to avoid problems, due to vibrations induced. For the slab track design, inserting elastic materials like elastomer layer is an effective method for reducing vibrations transmitted into the soil and to surrounding buildings (Fig. 1b-c). There are basically two most commonly used Floating Slab Track (FST) types, built within the unreinforced concrete layer. These are: FST sitting on a continuous elastomer layer-Type 1 (Fig. 1b) and FST sitting on spring units – Type 2 (Fig. 1c). The groundborne noise and vibration mitigation performance of a (FST) is determined by its first natural frequency. The vibration mitigation performance of a FST can be increased by lowering its resonance frequency; either by increasing the mass of the concrete slab or by increasing the resiliency of the elastic layer. Xin and Gao have investigated the problem of vibration transmission from a slab track structure into a bridge in a HSR by a theoretical analysis [2]. They have confirmed the great influence of stiffness of a slab mat on both rail and slab displacements, as well as on bridge and rail accelerations. There, total settlements have decreased and, but train accelerations also have suffered. Hence, many researchers and engineering institutions suggest optimizing the stiffness, by selecting an appropriate thickness of elastomer membrane, along with an appropriate choice of material [2]. However, in this study we did not consider to include such a slab mat layer, because of its minor impact on the total settlements.
The complexity of design of HSTR requires performance of a detailed analysis in order to maintain security and ride comfort in the trains and to avoid problems, due to vibrations induced in nearby buildings by waves transmitted through the soil. In recent years, there has been a huge expansion in the power and availability of numerical analysis techniques with a particular popularity of finite elements methods (FEM), with which both non-linear and linear elastic models are widely used in engineering practice. However, there are many researchers, who also favour to use soil structure interaction methods (SSI). In contrary to standard application of Elasticity modulus (E_s) and Poisson ratio (ν), the concept of modulus of subgrade reaction (k_s) is adopted. This widely used SSI concept refers to the relationship existing between soil pressure and deflection. It is applicable for various geotechnical problems, including continuous footings, mats, piles etc. k_s is described as the ratio of the increment of contact pressure ($\Delta\sigma$) to the corresponding change in settlement or deformation ($\Delta\delta$). Commonly plate load test is performed to obtain and plot values of σ versus δ to estimate a laboratory value of k_s. Results are usually non-linear and k_s needs to be obtained as a slope of either a tangent or secant line with preference of using the initial values. Thus, k_s is not an input parameter, but a determined value, whose magnitude must be calculated beforehand, based on current knowledge and available models. This fact was used subsequently by us, as one way of evaluating the accuracy of various subgrade models, where it is necessary to assume k_s as part of the analysis.

Bowles [3] has suggested that using SSI method over the FEM approach is preferable because of greater ease of use and more efficient computation time. Some scientists have tried to find a direct relationship between E_s and k_s. Terzaghi [3] was one of the first contributors, who attempted to make a correlation from plate load tests to estimate a numerical value of k_s. He has suggested an empirical method for k_s as an input value for the structural analysis of slab foundation design. However, the relationships found by Terzaghi are dependent on results from the plate load tests and they are exposed to size effects [3]. So the contribution by Terzaghi is mentioned only for historical reasons and it is not used in our analysis.
Earlier, Winkler (1867) has proposed a more specific formulation of the concept [4]. Thus, thanks to this significant contribution, the term 'Winkler foundation' or 'Winkler method' has been established. To describe this concept briefly; the coefficient k_s is transformed into a discrete spring element or support. In this concept of subgrade reaction, the foundation slab is assumed to act as an element, capable only of bending behaviour, as a plate or a beam [5]. The term 'elastic foundation' refers to the Winkler foundation model, and therefore analyses of this type are known as 'beams on elastic foundation (BOEF)' analyses. Although BOEF is widely used in geotechnical engineering practice, Winkler's assumptions cause some errors, as the model cannot transmit the shear stresses. It is because of the lack of spring coupling. Although various researchers have tried to deal with this limitation, those mitigation methods did not gain much popularity among majority of designers. Scientists like Filonenko and Borodich, Pasternak, Kerr and Hetenyi in their modified models found connectivity between individual Winkler springs by merging an elastic plate, which sustains some flexural or transverse shear deformations [6]. However during the settlement analysis, shear stresses do not play important roles, though in our study Winkler model was still employed. Numerical simulations of the static behaviour of a HSR were conducted in this study, but further dynamic and seismic analyses, together with numerical modelling of the vibration mitigation were not included.

2 Methodology

2.1 Soil-Structure Interaction

The soil-structure interaction (SSI) analysis evaluates the behavior of three linked systems. These systems include; the structure, the foundation and the soil underlying, as well as surrounding the foundation. The determination of the coefficient of subgrade reaction (k_s) is crucially important to obtain reliable results in the SSI concept. There are various relations proposed by some researchers in order to specify the k_s. The most common relations suggested for the coefficient of subgrade modulus are given in Table 1 [7].

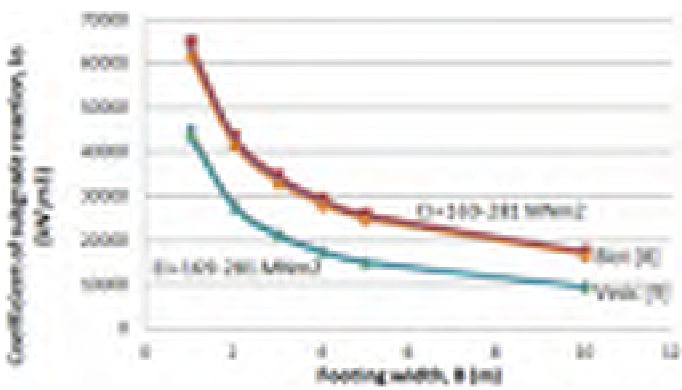

Figure 2 The variation of the coefficient of subgrade reaction depending on width and flexural rigidity of footing

In this study, Biot (1937) and Vesic (1961) findings are used, in order to determine the k_s [8, 9]. For various widths of footings (B) and various flexural rigidities of footings, k_s is calculated and these results are illustrated in Figure 2. According to these results, the k_s values of the Biot relation are greater than the k_s values of the Vesic relation for different values of widths of footings (B). With increase of B, k_s decreases significantly, whereas any significant increase in the k_s is not observed, despite substantial increases in flexural rigidity of footings.

Table 1 Common relations suggested for the k_s [7]

No.	Investigator	Proposed expression
1	Terzaghi	For sands $k_s = k_{s1}\left(\frac{B+1}{2B}\right)^2$ For clays $k_s = k_{s1}\frac{1}{B}$
2	Biot	$k_s = \frac{0.95E_s}{B(1-v_s^{\,2})EI}\left[\frac{B^4E_s}{(1-v_s^{\,2})EI}\right]^{0.108}$
3	Vlassov	$k_s = \frac{E_s(1-v_s)}{(1+v_s)(1-2v_s)}\left(\frac{\mu}{2B}\right)$
4	Vesic	$k_s = \frac{0.65E_s}{B(1-v_s^{\,2})}\sqrt[12]{\left(\frac{E_sB^4}{EI}\right)}$
5	Meyerhof and Baike	$k_s = \frac{E_s}{B\left(1-v_s^{\,2}\right)}$
6	Klopple and Glock	$k_s = \frac{2E_s}{B\left(1+v_s\right)}$
7	Selvadurai	$k_s = \frac{0.65}{B}\cdot\frac{E_s}{1-v_s^{\,2}}$
8	From Theory of Elasticity	$k_s = \frac{E_s}{B'\left(1-v_s^{\,2}\right)mI_sI_F}$

where:

E_s	modulus of elasticity of soil;	k_{s1}	the coefficient of subgrade reaction for a plate 1 ft wide;
v_s	Poisson’s ratio;		
B	width of footing;	I_s and I_F	influence factors which depend on the shape of footing;
EI	flexural rigidity of footing;		
μ	non-dimensional soil mass per unit length;	m	takes 1, 2 and 4 for edges, sides and center of footing respectively.
B’	least lateral dimension of footing;		

As seen in Table 1, the coefficient of subgrade reaction is related to the elasticity modulus of soil, which directly affects the coefficient of subgrade reaction. In order to accurately specify k_s values, it is required that realistic values of E_s of soil must be determined. The selection of the elasticity modulus of soils depending on only first soil layer beneath the footing will not give realistic results. Therefore, the effects of stratification on elasticity modulus of soil must be taken into consideration. Approach of an equivalent modulus of elasticity which includes the mechanical properties of soil layers within the influence depth is used in this study [7]. To explain this with respect to the Boussinesq theory, the contribution of external load to the increment of soil stress decreases with depth. Therefore, the upper layers have an important role for settlement of footings subjected to external loadings. Therefore, the equivalent elasticity modulus of soil (E_{eq}) is calculated by considering thicknesses of each layer and the depth factor (I_z) [7]. The equivalent elasticity modulus is calculated with the aid of Equation 1.

$$E_{eq} = \frac{\sum_{i=1}^{n} E_{si} I_{zi} H_i}{\sum_{i=1}^{n} I_{zi} H_i} \tag{1}$$

where:

E_{si}	elasticity modulus of soil at mid-point of each layer;
I_{zi}	depth factor at midpoint of each layer;
H_i	thickness of each layer;
n	numbers of layers.

The equivalent elasticity modulus of soil is calculated as 68144 kPa by using the data given in Table 2.

Table 2 Equivalent elasticity modulus parameters for High Speed Train Embankments

Number of Layers	Unreinforced Concrete Layer	Bearing Base Layer	Prepared Subgrade Layer	Subgrade Layer	Natural Subgrade Layer
E_{si} (kPa)	300000	120000	80000	60000	40000
H_i (m)	0,30	0,50	2,00	2,00	5,00
I_{zi}	0,99754	0,9918	0,9713	0,9308	0,73175

The coefficient of subgrade reaction is calculated by using both the Biot [8] and Vesic [9] relations, existing between E_s and k_s. The k_s values for these two relations are given in Table 3.

Table 3 The coefficient of subgrade reaction values for Biot [8] and Vesic [9] relations

Relation	k_s (kN/m³)
Biot [8]	36645
Vesic [9]	23480

As seen in Figure 2 and Table 3, the Biot [8] relation gives greater results than the Vesic [9] relation. Thus, it is clear that the total settlements of slab track will be lesser, when the Biot [8] relation values of k_s are used in the Winkler model. In other words, critical settlement values will occur, when the Vesic [9] relation values for the k_s are used in the Winkler model.

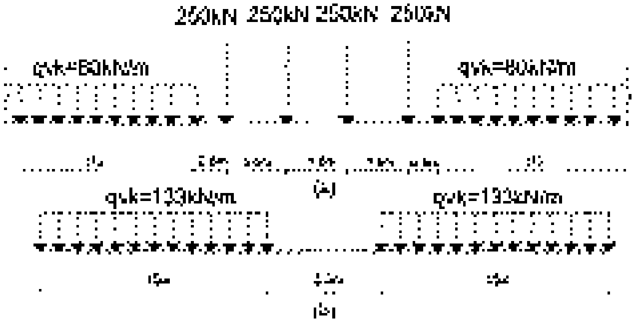

Figure 3 (a) Load Model 71, (b) Load Model SW/0 [10]

Another point is to determine external loadings on footings in the SSI concept. Rail traffic loads can be defined by means of load models. General rules about these load models are given for the static load condition of a High Speed Train load in the EN 1991-2 standards [10], where there are two different load models; as the Load Model 71 and the Load Model SW/0. These 2 load models represent the static effect of the vertical loads, due to normal HSR traffic. These load arrangements are shown in Figure 3a-b. In the EN 1991-2, the designated distance (1) varies, but it was taken as 20m in this study [10].

3 Results

Various load combinations are taken into account, while conducting settlement analyses. The total longitudinal settlement and contact pressure diagrams of HSR embankments for the model 71 and SW/0 are presented in Figures 4-5. Contact pressures obtained by Biot [8] and Vesic [9] relations are fairly the same. However, there is a slight difference in the total settlements computed by both approaches. The load combination SW/0 gives 10% greater contact pressures, than the load model 71. Contrary to the contact pressure diagrams, there is a significant difference in the total settlements by the Biot and the Vesic formulas. The reason of this different results lies with the k_s parameter, which is significantly greater for the Biot than the Vesic formula. The load model 71 gives 12% greater total settlements, than the load model SW/0 for the Vesic and 14% for the Biot formulas. For these investigated concepts, the total settlement does not exceed the limiting value of 0.01m per any 20m of embankment length, which is widely accepted criterion for any track design of an HSR in the Far East. Egeli and Usun [1] calculated the total settlement for the load model SW/0 by Plaxis (FEM) and obtained the value of 0.0075m, which is in agreement with our SSI analysis' results, obtained by using the Vesic equation.

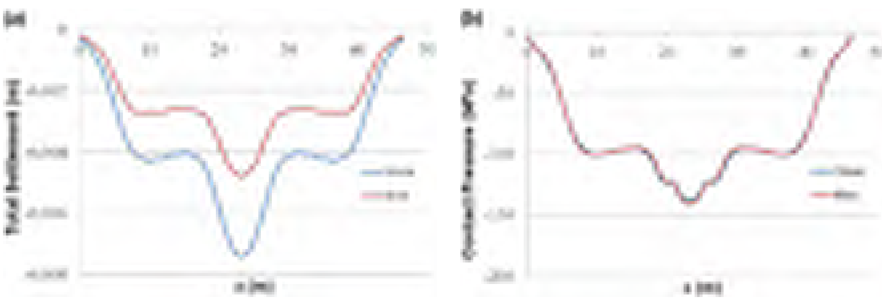

Figure 4 Calculations of (a) total settlement, (b) contact pressure, using the load model 71

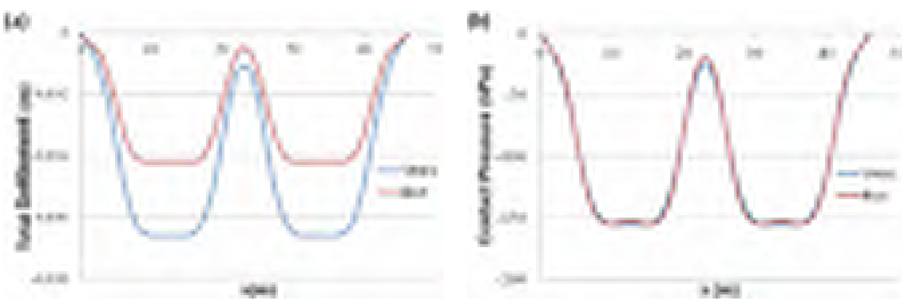

Figure 5 Calculations of (a) total settlement, (b) contact pressure, using the load model SW/0

4 Final conclusions

Numerical test methodology presented in this work and applied to the SSI model allows examining of the total settlements and contact pressures under a typical slab track subjected to HSR loads. Analyzed parameters (k_s, E_s, E_{seq}) estimate the amount of total settlements. Stratification effects of soil layers are taken into account thanks to considering the concept of equivalent elasticity modulus. In this study, it is proven that E_{eq} approach is a convenient tool to predict such critical total settlements. Numerical tests have been performed using different load arrangements and obtained results allow to evaluate the accuracy of different SSI approaches. Because implementation of the Vesic formula always produces the higher and more critical total settlements and the convergence of such results with the traditional FEM approach shows the usefulness of the approach used in engineering practice. Furthermore, the SSI concept has another distinct advantage above the standard finite element method, which is a timewise computation efficiency. Thus the study conducted has shown that the Floating Slab Track (FST) systems can be used successfully in HSR embankments without endangering passenger/load safety and with increased ride comfort. All these improvements have opened a new era in the analysis, and design of modern HSR infrastructures with variety of applications

References

[1] Egeli, I. & Usun, H.: Designing High-Speed Train Railway Embankments using Finite Element Analysis, The Arabian Journal for Science and Engineering, 37, Issue 8, 2012.

[2] Xin, T. & Gao, L.: Reducing slab track vibration into bridge using elastic materials in high speed railway, Journal of Sound and Vibration, 330, pp. 2237-2248, 2011.

[3] Bowles, J. E.: Foundation Analysis and Design, Fifth edition, The McGraw-Hill Companies, Inc. pp.501-523, 1996.

[4] Winkler, E.: Die Lehre Von Elasticitaet Und Festigkeit. First Edition, H. Dominicus, Prague, 1867.

[5] Hetenyi, M.: Beams on elastic foundation; theory with applications in the fields of civil and mechanical engineering, The University of Michigan Press, Ann Arbor, Michigan, 1946.

[6] Dutta, S. C. & Roy, R.: A critical review on idealization and modelling for interaction among soil-foundation-structure system, Computers and structures, 80, issues 20-21, pp. 1579-1594, 2002

[7] Sadrekarimi, J. & Akbarzad, M.: Comparative Study of Methods of Determination of Coefficient of Subgrade Reaction, EJGE, 14, 2009.

[8] Biot, M. A.: Bending of Infinite Beams on an Elastic Foundation, Journal of Applied Mechanics, Trans. Am. Soc. Mech. Eng., 59, A1-7, 1937.

[9] Vesic, A. B.: Beams on Elastic Subgrade and Winkler's Hypothesis, Proc. 5th Int. Conf. on Soil Mechanic and Foundation Engineering, Paris, pp. 845-850, 1961.

[10] EN 1991-2: Eurocode 1: Actions on structures – Part 2: Traffic Loads on Bridges, European Standard: CEN-Cenelec (www. cenelec.eu), Brussels, 1991.

DEFORMATIONAL PROPERTIES OF UNBOUND GRANULAR PAVEMENT MATERIALS

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Abstract

Selection of pavement materials takes very important role in pavement design procedure. The understanding of pavement structure behavior under cyclic loading is necessary in order to assure its serviceability during a predicted structure life. Unbound granular material layers in pavement structure represent a base for upper construction and their compaction and deformational behavior under cyclic loading have significant impact on the bearing capacity of upper layers and overall pavement construction. Unbound granular materials show complex behavior under cyclic loads with gradual accumulation of permanent deformation. Accumulation of a large number of small permanent deformation in unbound granular material usually affects on behavior of the sub-base layer and larger irreversible deformations in the upper layers of the pavement structure. Several analytical models have been developed to describe the development of the permanent deformations and behavior of unbound granular materials affected by these deformations. Recent studies and analyses are oriented on laboratory testing of different types of unbound materials under triaxial loading. These studies aim at developing analytical models which are more accurate in predicting deformation behavior of particular local materials under specific stress and moisture conditions.
The purpose of this paper is to present an overview of the models that describe permanent deformation in unbound granular materials and correlation between permanent deformation and number of load cycles. Particular emphasis in the analyses would be on models that can be applied on unbound pavement layers constructed of local materials used in Croatia.

Keywords: unbound granular material, pavement construction, permanent deformation, analytical models, triaxial loading

1 Introduction

Flexible pavement consists of one or more bound asphalt layer overlying one or more sub layers consisted of unbound granular material which are together compacted over a suitable soil subgrade and they are the most frequently used pavement type. These flexible road pavements deteriorate under effects of traffic load and weather over time. Because of that, selection of pavement materials takes very important role in pavement design procedure. The understanding of pavement structure behaviour under cyclic loading is necessary in order to assure its serviceability during a predicted structure life. Structure life of a pavement depends very much on the response and the quality of unbound granular layer material [1]. Unbound granular materials in sub-base and base layers have very important role in overall structural performance of the pavement and in general consisting crushed rock particles with certain amount of fines and different various water content [7]. Unbound granular layers in pavement structure have complex elastoplastic response under traffic loading. In this paper the idea is to present a short overview of the existing models for the description of performance of unbound layers and materials. Basic analyses of the factors influencing on that performance during time will be also presented.

2 Permanent deformations and appropriate behaviour models

Vehicles' wheeling causes repeated cyclic loads on pavement structures and two types of deformation, resilient and permanent deformations, occur (Fig.1.). Resilient deformations are recoverable part of deformations and they are related to the fatigue cracking of overlying asphalt layers. Permanent deformations are irreversible, very important accumulated plastic, non-recoverable deformations which may lead to rutting, one of the principal failure of flexible pavement structures. Accumulation of a large number of small permanent deformation in unbound granular material during each loading cycle usually can cause that failure or collapse of the sub-base layer and larger irreversible deformations like rutting in the upper layers of the pavement structure. These larger deformations accumulate during all service life [2,4,6]. Pavement design is made in the way that it can allow a very small number of a permanent deformation that does not threaten the stability of the pavements.

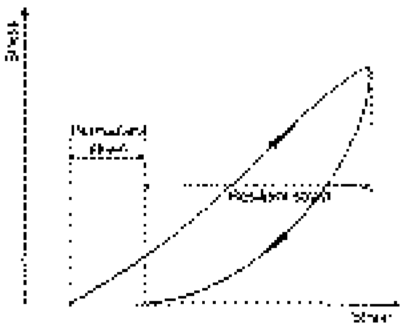

Figure 1 Strains in Granular Materials During One Cycle of Load Application [4]

Permanent deformation behaviour of unbound granular material is less studied than the resilient deformation because of influence of stress history on material behavior, difficulties in results of different laboratory test correlating, the complexity of the tests, so as complex and nonlinear constitutive relations [9, 10].
The most important and the best laboratory simulation of traffic loading to test the deformation properties is cyclic load triaxial testing [5]. Repeated load triaxial apparatus is device for cyclic load triaxial testing on prepared cylindrical specimen in order to investigate the mechanical behaviour of unbound granular material. Similar stress like in the field can be simulate with triaxial apparatus on prepared specimen – a cyclic vertical stress σ_1 and a cyclic horizontal stress σ_3 after that the corresponding deformations are measured. Unbound granular material under cyclic loading expose elastoplastic behavior by increasing of permanent deformation with load repetitions [7, 8, 9, 10]. Triaxial apparatus has a three controllers to generate back and air pressure and chamber in the partially saturated samples [1]. With repeated load triaxial test more information can be obtain from a single specimen because the experimental dispersion is reduced as is the amount of time and materials required [10].

2.1 Methods and models

The older method that is used for describing permanent deformations is empirical traditional method. Traditional design methods are based on experience with certain types of pavement material. That approach has a lots of disadvantages and limitations because of increases in traffic loads and growing transportation needs that resulted in rapid deterioration of road

networks. Because of that, nowadays, there is a strong desire to develop and study analytical methods and models to establish with more changes in loading and environmental conditions, construction techniques and materials [4]. Empirical models consider unbound granular material as linear elastic material with elastic moduli that is often determined on the empirical base [9]. Analytical models imply experimental approach application and mathematical characterization of behavior of unbound granular material under cyclic loading [3]. Models treat road pavement as a structure and they are based on analyzing the response of pavement layers and all structure under traffic cyclic load in certain environmental conditions [4]. The aim is to leave the traditional methods and develop an analytical model that can better describe permanent deformation behaviour. Analytical approach requires better understanding of the behavior and properties of the materials. A large number of tests in laboratories can enable development of models that can predict the permanent deformations. In laboratory it is possible to simulate the properties of material and response of unbound granular materials under cyclic traffic loading. Analytical pavement models can correlate deformations with layer thicknesses, layer material, conditions, traffic load which enable a prediction of permanent strain value at any loading cycle at any stress [2, 7].
Resilient response (Fig. 1.) is non-linear and these deformations can be interpreted using several models like Uzans and k-θ models [3, 5]. Irreversible, permanent deformations and strains are more complex and on their behaviour affect many factors such as stress, number of load cycles, grain shape, stress history, density, mineralogy, water/moisture content, fines content, etc. [5].

2.1.1 Impact of stress

Stress has a significant influence on the permanent deformations in unbound granular layers. Stress state similar to the stress in the field is simulated in triaxial apparatus. One of first significant conclusion that accumulation of axial permanent strain is related to deviator stress and confining pressure is given by Morgan (1966) [4].

Table 1 Equations that describe relationship between permanent strain and stress

Barksdale [13]	$\varepsilon_{1,p} = \dfrac{q/K \times \sigma_3^{\,n}}{1-\left[\dfrac{(R_f \times q)/2(C \times \cos\phi + \sigma_2 \times \sin\phi)}{(1-\sin\phi)}\right]}$
Pappin [15]	$\varepsilon_{s,p} = (f\mathrm{nN}) \times L \times \left(\dfrac{q^0}{p^0}\right)_{max}^{2.8}$
Paute [14]	$A = \dfrac{\dfrac{q_{max}}{(p_{max}+p^*)}}{b \times \left(m - \dfrac{q_{max}}{(p_{max}+p^*)}\right)}$

Barksdale [13] relates the permanent strain to the ratio of repeated deviator stress and constant confining pressure with equation presented in Table 1. Expression depends on apparent cohesion C, angle of internal friction ϕ, constants K and n and constant relating the compressive strength to an asymptotic stress difference R_f. Pappin [15] also gives expression for accumulated permanent strain that depends on shape factor fnN, modified deviator stress q^0 and modified mean normal stress p^0, stress path length L. He also declares that the large permanent strains did not occur unless the material was stressed close to the static failure limit. At the University of Nottingham an experiment on five different specimens (aggregates) was carried out and none of listed equations and models seemed to give a satisfying explanation of the influence of stresses on accumulation of permanent deformation [7]. It was estimated that the non-linear model proposed by Paute (Table 1.) [14] the best fits the data at low levels of shear stress ratio when the material have almost total resilient behavior. The model is weak for high shear stress ratios when the accumulation of permanent deformations is progressive and cause deterioration [7]. Paute defines the hyperbolic expression for limit value of maximum permanent axial strain (A). Limit value A increases when the maximum shear stress ratio increases and it depends on parameters m (slope of the static failure line) and p* (stress parameter defined by intersection of the static failure line and the p-axis in p-q plane). It also depends on slope of static failure line m, maximum mean normal stress p_{max} and regression parameter b [14].

2.1.2 Effect of number of load applications, cycles

Some researchers [11-14] have followed another one approach trying to relate the value of permanent deformation with a number of load applications. Number of load traffic cycles is very important factor for deterioration of pavements. Cyclic traffic loading is also very often simulated in laboratories using triaxial apparatus test. In Table 2. is presented an overview of analytical models that offer good expression of the correlation between the strain and number of loading cycles [11-14].

Table 2 Equations that describe relationship between permanent strain and number of load cycles

Barksdale [13]	$\varepsilon_{1,p} = a + b \times \log(N)$
Sweere [12]	$\varepsilon_{1,p} = a \times N^{b}$
Khedr [11]	$\frac{\varepsilon_p}{N} = A \times N^{-m}$
Paute et al. [14]	$\varepsilon_{1,p} = A \times \left(1 - \left(\frac{N}{100}\right)^{-B}\right)$

Every equation is obtained on the bases of triaxial test results on differently prepared specimens. Barksdale [13] proposed an equation after tests with 10^5 load applications where strain depends on number of cycles N and a and b, constants for a given level of σ_1- σ_3 and σ_3. Swerre [12] proposed a non-linear model which depends on regression parameters a and b after 10^6 load cycles (N). Sweere's model equation is based on Korkiala-Tanttu's model (Eqn. (1) [16].

$$\varepsilon_p(N) = CN^b \frac{R}{A - R} \tag{1}$$

Difference between the Korkal-Tanttu's and Sweere's equations is in an effect of stress levels based on classic soil mechanics laws. C and b in eqn (1) are material parameters and R is shear stress ratio q/q_f., deviator stress/deviator stress at the failure [8]. Khedr [11] presented the model with conclusion that the rate of permanent strain accumulation decreases logarithmically with number of cycles where m is material parameter and A is material and stress-strain dependent parameter given as a function of shear stress ratio and resilient modulus. Together with previously described studies that examined influence of stress on permanent strain accumulation an experiment that studied effect of load cycles on the same five specimens was carried out. Paute's non-linear model [14] fit the data the best and showed the least error in most cases in relation to others models that depend on number of load cycles [7]. Paute's model depends also on regression parameters A and B after $8x10^4$ load repetitions.

2.1.3 Effect of moisture content

Moisture is mostly related with the fines content of the unbound granular material. Adequate amount of water doesn't have a negative influence on unbound granular material behavior, but in combination with rapidly applied loads lied to high pore pressure and saturation. The effective normal stress is reduced by that pore pressure generation and can lead to the occurrence of permanent deformation. Haynes, Barksdale, Dawson and others concluded that combination of low permeability and high degree of saturation lead to lowering effective stresses and high pore pressure generation that lead to the lowering deformation resistance and stiffness [4].

2.1.4 Others Effects possible impacts

Other parameters that effect on deformations are stress history (Brown and Hyde 1975), density (Barksdale 1972, Allen 1973, Holubec 1969), fines content (Barksdale 1972, 1991, Thom and Brown 1988), aggregate type (Allen 1973), grain shape (Barksdale and Itani 1989, Janoo 1998), surface texture and mineralogy. Some models also give a correlation between static and cyclic loading tests (Gerrard 1975, Lentz and Baladi 1981) and correlation between resilient and plastic behavior (Veverka 1979 [5, 10].

3 Local source for unbound granular layers in Croatia

The aggregate type and mineralogy of origin are very important parameters for unbound granular materials. Crushed rock have to be resist on crushing under traffic loading. Unbound granular layers are mostly made of local stone materials. The construction of pavement structure can be accelerated if local materials are used. In Croatia, the aggregate used for pavement base and subbase is a crushed limestone aggregate that is local material with a good quality and parameters required in Technical Specifications [17].

Lekarp and Dawson studied the permanent deformation behaviour of five different aggregates and one of these was crushed limestone. Test that depends on number of cycles load show that Paute's model (Table 2.) the best fit the data. Test related to the effect of stress also shows that Paute's model (Table 1.) is the most appropriate model for limestone materials [6]. Khedr [11] was also studying crushed limestone material under repeated loading in triaxial apparatus. The conclusion was that the rate of permanent strain accumulation decreases logarithmically with the number of load repetitions according to the equation:

$$\frac{\varepsilon_p}{N} = A \times N^{-m} \tag{2}$$

where:

m material parameter;

A material and stress-strain parameter given as a function of shear stress ratio and resilient modulus.

At the University of Perugia and Marche Polytechnic a crushed limestone aggregate through was investigated under repeated loads in triaxial tests. They made tests on two mixtures: one was typical mixture for unbound layers according to the Italian Technical Specifications and the second was combination of the first with addition of a silty clayey soil fraction [2]. The results were analyzed and compared with a few known models proposed from Barksale, Sweere and Paute given in Table 2. Also is given a new linear-exponential model [2] is prosed as:

$$\varepsilon_p(N) = A + B \times N \tag{3}$$

where:

A constant expressing the variability of permanent strain accumulated during the first load cycles;

B strain rate per load cycle.

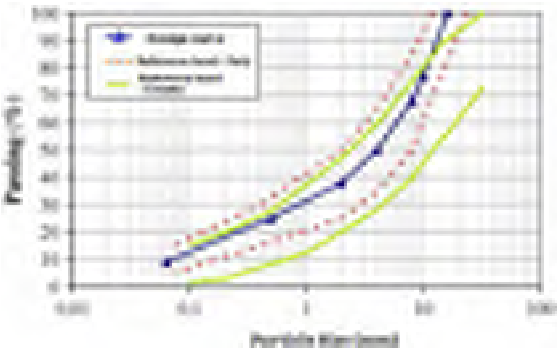

Figure 2 Design gradation for tested materials in reference band from Italy and Croatia [2, 17]

The determination coefficients R^2 which are obtained for every model provide that the highest value have a last model – greater than 0,95 [2]. Gradation of materials that have been tested are located in reference band in Italy but also in reference band in Croatia (Fig. 2.) and given model can be used for Croatians unbound layers although some more detailed analyses will help in estimating better permanent deformation behaviour in crushed limestone material.

4 Conclusion

In this paper an general overview of the most used models for the determination of permanent deformation behaviour in unbound pavement layer is presented. It can be established that the impact of stress and the number of load cycles have the most important effect on unbound granular material deformation behavior. Each model also has different influence factors such as cohesion, angle of internal friction, different material constants, shape factor, stress path length and regression parameter. Models that relate the permanent deformation with the number of load cycles mostly depend on the number of load cycles. Every model has several parameters like regression, material, stress-strain parameter which also have a major impact on deformation behavior. Results in the mentioned experiments that studied the impact of stress and the number of load cycles shown that Paute model fits the data best.
Crushed limestone is the mostly used local material in Croatia for pavement structures. In this paper an overview of a few analytical models that describe deformation behavior in that material like Paute's and Khedr's model are presented. Also, a linear- exponential model is given that can be used for Croatian unbound pavements' materials because that model the best fits the data with the determination coefficient greater than 0,95.
Analytical models are very complex because of the effect of a lot of parameters that impact on deformation behavior of unbound pavements' materials. It is necessary to conduct detailed analyses for crushed limestone locally used in Croatia in order to establish detailed models for the pavement constructions' design.

Reference

[1] Perez, I., Medina, L. & Romana, M.G.: Permanent deformation models for a granular material used in road pavements, Construction and Building Materials, 20, pp. 790-800, 2006.

[2] Cerni, G., Cardone, F., Virgili, A. & Camili, S.: Characterisation of permanent deformation behaviour of unbound granular materials under repeated triaxial loading, Construction ann Building Materials, 28, pp. 79-87, 2012.

[3] Cuculić, M., Arbanas, Ž. & Deluka-Tibljaš, A.: Deformacijske značajke nevezanih zrnatih materijala, Zbornik radova, Građevinski fakulet, 12, pp. 232-253, 2009.

[4] Lekarp , F., Isacsson, U. & Dawson, A.: State of the Art. II: Permanent strain response of unbound aggregates, Journal of transportation engineering, 126/1, pp. 76-83, 2000.

[5] Uthus, L.: Deformation Properties of Unbound Granular Aggregates, Thesis for the degree of philosophiae doctor, NTNU, Trondheim, 2007.

[6] Lekarp, F. & Dawson, A.: Analysis of permanent deformation behaviour of unbound granular materials, International Symposium on Thin Pavements, Surface Treatments, Unbound Roads, Canada, 1997.

[7] Lekarp, F. & Dawson, A.: Modelling permanent deformation behaviour of unbound granular materials, Construction and Building Materials, 12, pp. 9-18, 1998.

[8] Erlingsson, S. & Rahman, M.S.: Evaluation of Permanent Deformation Characteristics of Unbound Granular Materials from Multi-Stage Repeated Load Triaxial Test, TRB, 2013.

[9] Hornych, P. & Absamad, A: Selection and Evaluation of Models for Prediction of Permanent Deformations of Unobund Granular Materials in Road Pavements, Publication SAM-05-DE10. s.l., Competitive and Sustainable Growth (GROWTH) Programme, 2004.

[10] Gidel, G., Hornych P., Chauvin, J.J., Breysse, D. & Denis, A.: A new approach for investigating the permanent deformation behaviour of unbound granular material using the repeated load triaxial apparatus, Bulletin de Liaison des Laboratoires des Ponts et Chaussees, 233, pp. 5-21, 2001.

[11] Khedr, S.: Deformation characteristics of granular base course in flexible pavement, Transportation Research Record, 1043, pp. 131-138, 1985.

[12] Sweere, G.T.H.: Unbound granular bases for roads, PhD Thesis, University of Delft, 1990.

[13] Barksdale, R.: Laboratory evaluation of rutting in base course materials, Proceedings of the Third International Conference on Structural Design of Asphalt Pavements, London, pp. 161-174, 1972.

[14] Paute , J., Hornych, P. & Benaben, J.: Comportement mecanique des graves non traitees, Bull Liasion Laboratories Ponts Chaussees, 156, pp. 21-36.1988

[15] Pappin, J.W.: Characteristics of a granular material for pavement analysis, PhD. Thesis, Department of Civil Engineering, University of Nottingham, 1979.

[16] Korkiala-Tanttu, L.: A New Material Model for Permanent Deformations in Pavements, Proceedings of the 7th Conference on Bearing Capacity of Roads and Airfields, Trondheim, Norway, 2005.

[17] Opći tehnički uvjeti za radove na cestama, Hrvatske ceste – Hrvatske autoceste, Zagreb, 2001.

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APPLICATION OF NEURAL NETWORKS IN ANALYZING OF ROCK MASS PARAMETERS IN TUNNELLING

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Abstract

One of the key problems in tunnelling is to define realistic parameters for rock mass properties as a basis for successful numerical modelling. The main goal of this problem is how to extrapolate the parameter from the zone of testing to the whole area (volume) that is of interest for interaction analyses of the system rock mass-structure. So it is necessary to find an appropriate way to find intelligent tools to combine data from empirical classification rock mass methods having in mind that there are a lot of variations in statistic values.
First step in the procedure is to divide the tunnel length in quasi-homogenous zones, while the second is to define adequate geotechnical and numerical models as a basis for interaction of rock – structures system and stress-strain behaviour of rock massif. Artificial neural networks (ANN) have been found to be powerful and versatile computational tools for many different problems in civil engineering over the past 2 decades. They have proved useful for solving certain types of problems, which are too complex, or too resource-intensive nonlinear problems to tackle using more traditional computational methods, such as the finite element method. ANN-s are intelligent tools, which have gained strong popularity in a large array of engineering applications such as pattern recognition, function approximation, optimization, forecasting, data retrieval, automatic control or classification, where conventional analytical methods are difficult to pursue, or show inferior performance. A review of some problems in tunnelling that were successfully solved by using neural networks is presented in this paper. A general introduction to neural networks (NN), their basic features and learning methods is given. After that, it is possible to use neural networks to solve necessary problems in tunnelling.

Keywords: tunnelling, rock complexes, classification, extrapolation, neural networks

1 Introduction

Neural Networks have made a remarkable contribution to the advancement of various fields of endeavor and have become very popular for data analysis over the past 2 decades. Their application in the civil engineering field is considered in this paper. Neural networks are intelligent systems that are based on simplified computing models of the biological structure of the human brain, whereas the systems based on traditional computer logic require comprehensive programming in order to perform a given task. Artificial Neural Networks (ANN-s) are suitable for multivariable applications where they can easily identify interactions and patterns between inputs and outputs. ANN models do not require complicated and time consuming finite element input file preparation for routine design applications. They are able to infer important information for the task, which is being solved by them, if data that is representative of the underlying process to be implemented, is provided. Neural networks have a self-learning ability, which is particularly useful where comprehensive models that are required for conventional computing methods are either too large or too complex to represent accurately, or simply doesn't exist at all.

2 Artificial Neural Networks – general overview

The functioning and the survival of intelligent systems depends on their system for processing information. The nervous system is the system for processing information in biological systems. It consists of the brain, as the central processing system for processing information and a set of sensors. The basic information element in biological systems is the neuron or the neural cell. There are billions of such processors in the brain. They are distributed, work simultaneously and cooperate. The neuron processors make the microstructure, the material basis of the biological intelligence. An approach to research of artificial intelligence is motivated by this observation and deals with the concepts of artificial neuron and artificial neural network, as the microstructure of artificial (synthetic) intelligence. Artificial neurons are inventions that are inspired by the anatomy and physiology of biological neurons. Figure 1 presents the morphology of a human neuron. It is assumed that the neuron is of an electrical nature – it has an electrical potential in respect to the environment. This electrical potential is changed due to external influences. The neuron has several inputs through which it receives electrical impulses, and only one output through which it sends an electrical impulse in the environment.

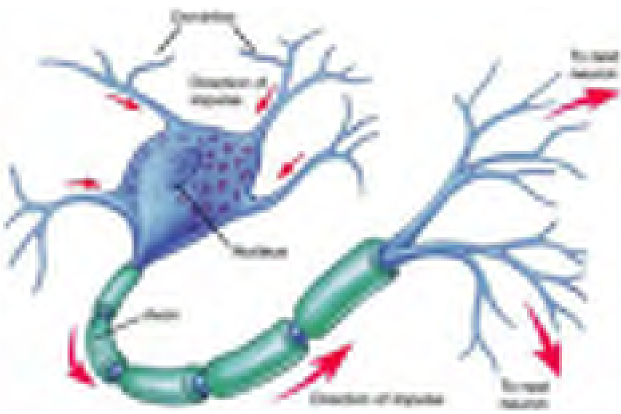

Figure 1 Example of a neuron morphology

Modeling of a neuron can be a very complex task. It involves the following key notions:
- body of the neuron (soma);
- axon – output neural strand through which it sends signals to other neurons;
- dendrites – growths of the soma which form a dendritic canopy;
- synapses – places in dendrites or soma where the neuron receives signals from other neurons; synapses are connections with other neurons;
- cumulative postsynaptic potential (SPP) is a cell potential, which is formed due to the cumulative synergistic influence of synaptic potentials;
- synaptic influence (weight): synapses have different influence on the formation of SPP. Some of them have a positive influence (excitation), while some have a negative effect (inhibition). Every synapse has its “weight” in the formation of SPP;
- threshold of the neuron excitation: minimal SPP needed for the triggering of an output signal from the neuron;
- presynaptic processing: processing of the biosygnals by direct connection of the axon with the synapses, before the potential is transmitted to the soma of the neuron;
- neural network: the structure of the connected neurons.

3 Artificial Neural Networks – general overview

Software NeuralTools ver. 6 provides a tabular view of all data analysis. Initially, it is necessary to define the variables and cases in which you need to model the problem. The values of variables are grouped in columns and name of them are written in the first row of the table. Each cases is represented by a series of table and is composed of a group of independent variables and known (or unknown) value of the dependent variable output . The goal is to predict the values of output variables for the cases for which they are unknown. To solve the given assignment problem, all data were grouped into two separate tables: the data for training and testing the neuronal network and data to predict the value of the output variable. Adopted the following independent variables:

- Bulk density (volume weight) γ;
- Compressive strength σ_p;
- Ingress of water;
- RQD – Rock Quality Designation;
- Average distance between leak (crack) Ls;
- Seismic velocity Vp.

Neuronal network has one output variable whose value depends on the input variables:

- RMR – Rock Mass Rating.

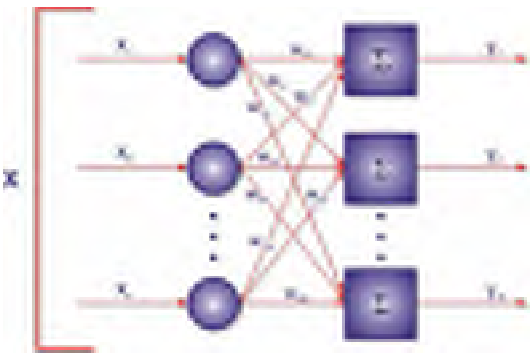

Figure 2 Model onelayer artificial neural network

Figure 3 The neuron activation function

To solve the given problem were analysed 97 cases, of which 78 (80%) belong to the group of cases for training and the rest were used for testing the network.

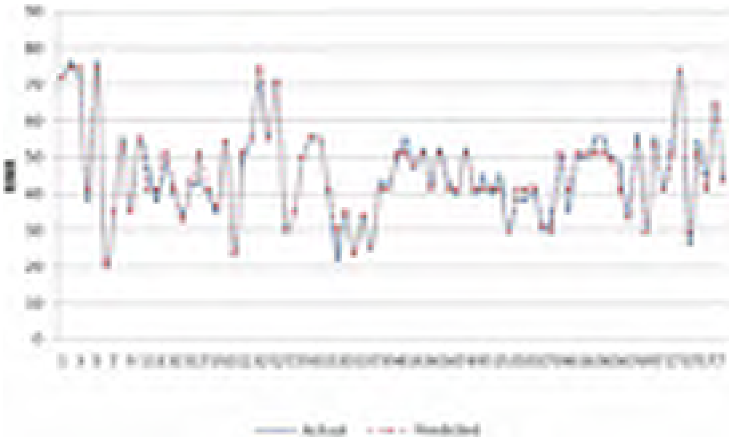

Figure 4 Cases of training

It may be noted that the trained neural network predicts a good quality rock masses RMR for inputs belonging to the interval sizes with which it was trained, or training for the 78 cases analyzed. The following figure shows a histogram of actual and predicted values for training cases and the frequency of their recurrence. Most of the residual values are around 0 which is a good indicator of the accuracy of the model for predicting RMR.

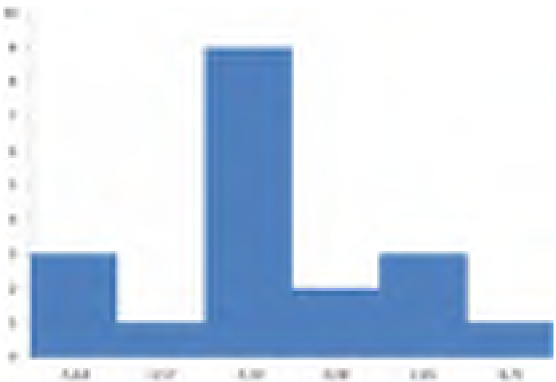

Figure 5 Histogram of residual values of training cases

Table 1 Residual values for test cases and their frequency of repetition

Residual value	Frequency of repetition
-7,030006814	3
-5,165334925	1
-3,300663036	9
-1,435991147	16
0,428680742	26
2,293352631	14
4,15802452	6
6,022696409	3
	Σ = 78

Neuronal network testing was performed on 19 data set that were not involved in the training of the network. Comparison of predicted RMR values obtained using the trained neural network and the expected results of the analyzed test cases is presented in follows figure.

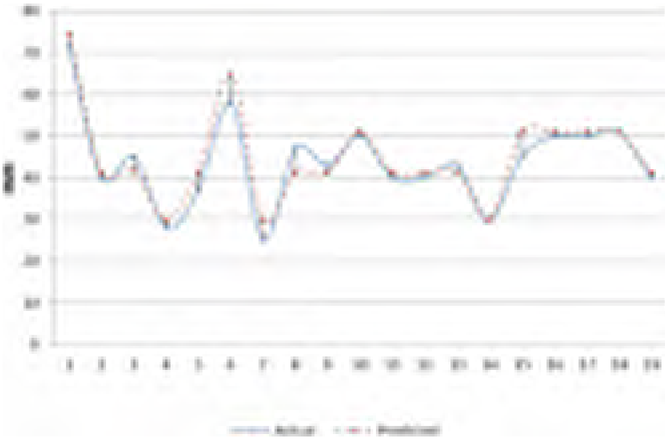

Figure 6 Graphic review of actual and predicted values for RMR test cases

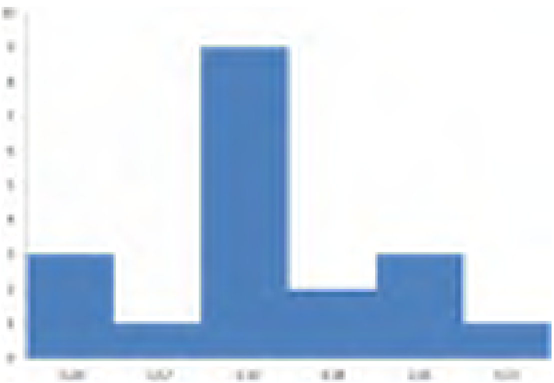

Figure 7 Histogram of residual values of testing cases

Testing the trained neural network confirms the fact that the model allows qualitative forecasting RMR on one side of the input data that were not used in the training of the network, for 19 cases of testing.

Table 2 Residual values for test cases and their frequency of repetition

Residual value	Frequency of repetition
-5,6423958	3
-3,5696133	1
-1,4968307	9
0,5759518	2
2,6487343	3
4,7215168	1
	Σ = 19

4 Conclusion

Although the first information about neural networks dates back to 1940, their practical application begun only four decades later, after discovery of appropriate algorithms which significantly increased their applicability. A lot of research is currently conducted in the sphere of neural networks, and these networks are increasingly studied at many universities all over the world. Neural networks are an example of a sophisticated modeling technique, and they have found their practical application in different areas, namely as a method for solving a variety of difficult and complex engineering problems. The application of neural networks for prognostic modeling aimed at predicting Rock Mass Rating – RMR is highly significant for the construction design process. Most experimental models are extremely expensive, while analytical models are quite complicated and time consuming. That is why a modern type of analysis, such as modeling based on neural networks, can be considered as extremely helpful, especially in cases when some prior analyses had already been made.

References

[1] Baum, E.B.: Capabilities of Multilayer Perceptrons, Journal of Complexity, vol. 4, 1988.

[2] Baum, E.B., Haussler, D.: What Size Net Gives Valid Generalization, Neural Computation, vol. 1, 1989.

[3] Chen, A. M., Lu, H.M., Hecht-Nielsen, R.: On the geometry of feed forward neural network error surfaces, Neural computations, pp. 910-927, 1993.

[4] Leshno, M., Lin, V.Y., Pinkus, A., Schocken, S.: Multilayer Feed forward Networks with a Nonpolinomial Activation Function Can Approximate any Function, Neural Networks, vol.6, 1993.

[5] Vouk, D., Malus, D. and Carević, D.: Neural networks and their application in water management, Građevinar 63 (2011) 6, pp. 547-554

[6] Žagar, Z., Janjuš, G.: "Use of neural networks in the design of wooden structures", Građevinar 54 (2002) 10, pp. 577-583

[7] Knežević, M.: Risk management of civil engineering projects, doctoral dissertation, Civil Engineering Faculty, University in Belgrade, Serbia, 2005.

[8] Knežević, M., Zejak, R.: Neural networks – application for usage of prognostic model of the experimental research for thin reinforced-concrete columns, scientific research work, Materials and constructions, 2008.

[9] Grossberg, S.: Classical and instrumental learning by neural networks, Progress in theoretical biology, vol. 3, pp. 51–141, New York: Academic Press, 1974.

[10] Rumelhart, D.E., Hinton, G.E., Williams, R. J.: Learning internal representations by error propagation, Parallel distributed processing, vol. 1, pp. 318-62. Cambridge, MA: MIT Press, 1986.

[11] Lazarevska, M., Knežević, M., Cvetkovska, M., Ivanišević, N., Samardzioska T., Trombeva-Gavriloska, A.: Fire-resistance prognostic model for reinforced concrete columns, Građevinar 64 (2012) 7, pp. 565-571

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DETERMINATION OF BLAST INDUCED DAMAGE ZONE DURING TUNNEL EXCAVATIONS IN CARBONATE ROCKS

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Abstract

Tunnel excavation by blasting inevitably results in the rock mass damage around the excavation profile. The rock mass damage immediately next to the tunnel profile emerges as the expanding of the existing cracks and the appearance of new cracks, i.e. as the change of the physical and mechanical properties of the rock mass. Measurements were obtained from the surface and from the tunnel opening by using geophysical and visual methods.

Keywords: blast induced damaged zone, underground excavation, in situ investigations, rock mass physical and mechanical properties

1 Introduction

In underground structures, unlike in other structures, the ground (soil and rock mass) acts not only as the loading mechanism, but as the primary supporting medium as well [1]. When an excavation is made, if the ground respond elastically, the strength of the ground keeps the hole open until support elements are installed. Even after support is in place, the ground provides a substantial percentage of the load-carrying capacity. The rock mass surrounding an underground structure is a construction material and its characteristics are as important as those in other aspects of civil engineering.

When an underground object is excavated in rock mass, the equilibrium state within a rock mass prior to the excavation is disturbed and the material particles surrounding the opening have to resist that pressure as an excess pressure which before was supported by the excavated material. The excavation boundary becomes a principal stress plane with one of the principal stresses (of zero magnitude) being normal to the surface, which causes a major disturbance of the pre-existing stress field in the principal stress magnitudes and their orientations [2]. The original stress field in a rock mass is deviated by the opening of a cavity and channelled around it to create a zone of increased stress deviator around the walls of the excavation. This natural phenomenon of channelling of stresses around the cavity (an arch effect) ensures the stability and the duration of the cavity [3]. Another effect of excavation is that any fluid pressure existing in the rock mass prior to the excavation will be reduced to atmospheric pressure at the boundary of an excavation open to the atmosphere, so any fluid within the rock mass will tend to flow into the excavation.

When an underground excavation is carried out by drilling and blasting, it inevitably results in the damage of the rock mass around the excavation profile [4]. The damage is manifested in the form of opening the pre-existing fractures, creation of new fractures and redistribution of stresses [4, 5, 6]. Consequences of these processes are considerable changes in physical, mechanical and hydraulic properties of the rock mass. The region in which irreversible changes to the physical, mechanical and hydraulic properties of the rock mass occur is referred to as Blast-Induced Damage Zone (BIDZ) [4, 7].

Figure 1 Schematic view of Blast-Induced Damage Zone (BIDZ)

Since the stability of the underground structure is dependent upon the integrity of the rock mass immediately surrounding the excavation, it is a usually considered that the presence of the BIDZ can seriously affect the stability and performance of an underground excavation. However, this consideration is largely based on methods related to overbreak (which represents only a part of the damage zone), rather than accounting for the actual features of the damage [4, 7]. To be able to assess the significance of the BIDZ and its influence on the performance and stability of an excavation, the mechanical properties of this zone must be understood, especially the inherent properties of damaged rock such as its strength and stiffness [4, 7]. The importance of BIDZ depends on the sort, purpose and the required life span of an object, and BIDZ is present during the following works:

- surface and underground excavations in the exploitation of mineral resources;
- excavations in civil engineering (construction of roads, geotechnical and hydro-technical objects, foundation pits, road, railway and road tunnels);
- excavations for deep underground repository for dangerous materials (nuclear, medical and chemical waste, oil and gasses).

2 State of the art

In spite of the fact that many of the present and future major civil engineering underground projects are planned and executed in jointed carbonate rocks, only a few studies for civil engineering structures (road tunnels, railway tunnels, hydropower caverns, underground storage caverns) have considered the influence of excavation by drilling and blasting on the extent and properties of the BIDZ. Most of the investigations of BIDZ were carried out in hard rocks such as granites and gneisses, which are completely different in terms of physical, mechanical and hydraulic properties from carbonate rocks, so that the extent of the BIDZ and its influence on the excavation contour stability, support costs and safety of the workers and equipment are much smaller than those in carbonate rocks. In BIDZ research for nuclear waste repositories, BIDZ is treated from the perspective of potential path for the nuclear waste leakage, so only the hydraulic properties of BIDZ were the main subject of the research. Important research of rock mass damage around the excavating profile was done by Kujundžić in 1970's. In 1979 the measurement results using the cross–hole method in gallery tunnels 5 meters in diameter, during the construction of the Martinje dam were published.

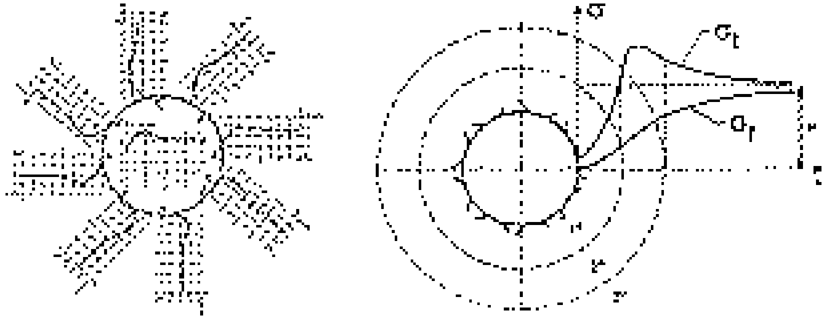

Figure 2 Graphical presentation of v_p wave velocity changes from the tunnel boundary and the characteristic zones around the excavation profile [6]

The measurements showed changes in the values of v_p wave velocities from the edge of the profile to the rock mass, and the authors defined three characteristic zones around the tunnel profile: the loosened zone (with lowest velocities), the stress bearing ring (with highest tangential stress and velocities) and the uninfluenced zone (with declining velocities and background stresses), Figure 2.

Capozza has made similar research to determine the damage zone around the excavation profile in a tunnel by using seismic cross–hole method [8]. The measurements consisted of P wave velocity changes in the first several meters from the profile edge (Figure 3). The values of P wave velocities in the vicinity of the excavation contour were reduced by 25-50% as compared to a more distant rock mass, so that the depth of the rock mass damage was three meters in the tunnel located at the depth of 2100 m.

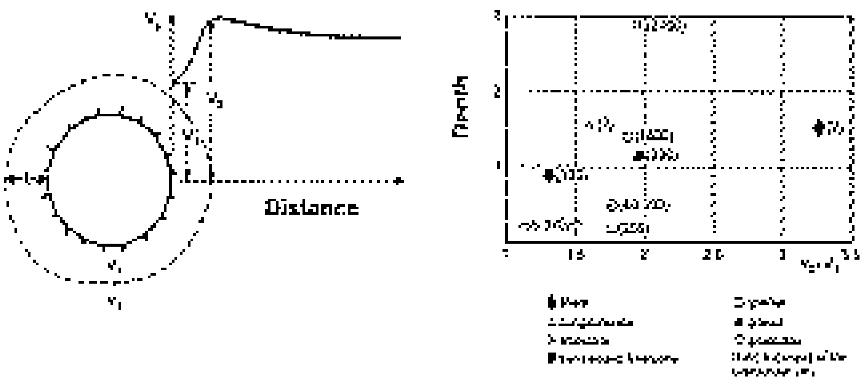

Figure 3 Graphical presentation of damage zone depth around the tunnel profile by comparing the propagation P wave velocities in different materials [8]

Investigations done in an iron mine in Sweden by using the cross–hole seismic method, spectral analysis of surface waves SASW and borehole video prospection (BIPS) resulted in blast damage zone depth of 0.5 to 1 m in eruptive rocks [9].

A recent significant research of BIDZ in carbonate rocks has been done on several road tunnels in Croatia by geodetic recording of the excavation profile, visual inspection of trial boreholes by means of a video camera, seismic cross-hole tomography, measuring the ground vibrations and recording the blasting parameters [4]. The damage zone was measured in carbonate rocks, and the detected damage depth was up to 3 m. This study has shown a potential influence of BIDZ on the stability of the excavation contour and increased construction costs, and has indicated the need to further study this phenomenon.

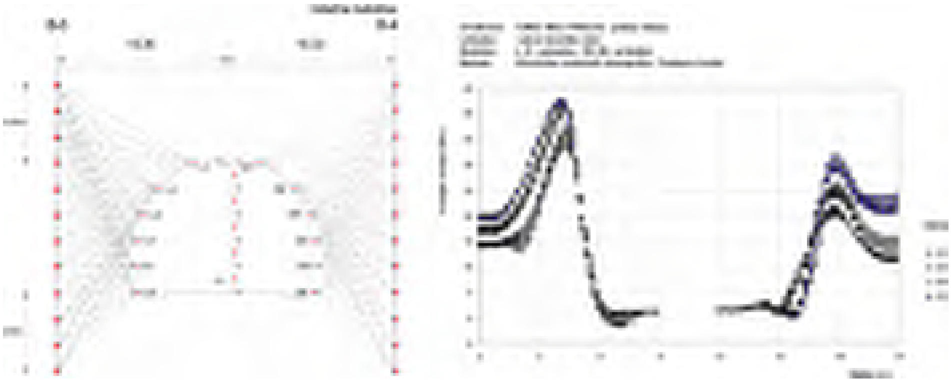

Figure 4 Rock mass stiffness reduction around the tunnel excavation profile detected by seismic refraction [4]

The in-situ methods of BIDZ investigation applied so far used the principle of drilling core mapping and registration of blasting parameters [10, 11], drilling core mapping and measuring peak particle velocities [12], measuring the visibility of traces of contour blastholes, overbreak and peak particle velocity [11, 13, 14, 15], applying visual methods [4, 9, 16, 17] and applying geophysical methods [4, 5, 6].

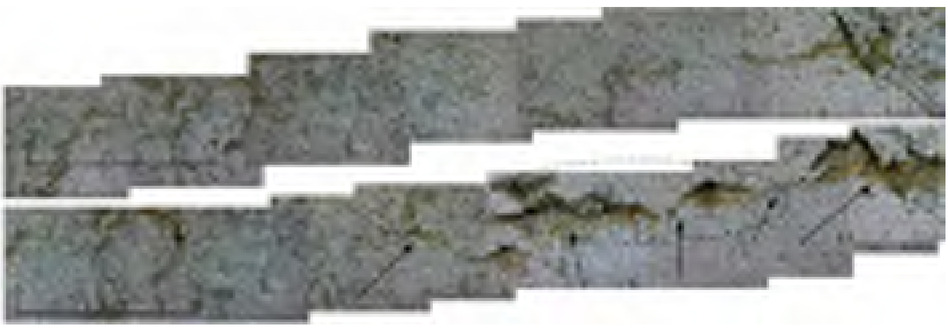

Figure 5 Borehole video prospection of rock mass damage around tunnel profile before and after blasting [4]

According to available literature, there does not seem to be a common opinion in selecting adequate research methods to define, classify and characterize the BIDZ [4]. Besides, the majority of the damage zone research has been done on laboratory models, which cannot properly estimate in situ rock mass conditions.

3 Proposed methodology of research

Regarding the complexity of the phenomenon, the research of BIDZ must be characterized by a multidisciplinary approach, i.e. the cooperation of different branches of science (civil engineering, mining, geology, geophysics and geodesy), which has not been applied in any research so far. The multidisciplinary approach includes in situ and laboratory research, analysis and interpretation of the results and numerical modelling. In situ investigations of an underground object should be planned in each geotechnical unit (zone with uniform rock mass properties) in two phases:

1 prior to drilling and blasting – measurements from the surface and from the tunnel profile;
2 after drilling and blasting – combined measurement from the tunnel profile and from the surface and measurements after placing a primary support.

The purpose of in situ measurements in two stages is to compare the state of the rock mass around the underground excavation contour before and after the excavation, i.e. to detect the BIDZ by the changes in the physical and mechanical properties of the rock mass.
During in situ research, one should consider specific character of the underground space, such as difficult accessibility from the surface, limited working space, poor illumination and ventilation and other obstructions (electrical installations, water, machine vibrations due to transport, fan noise), which can negatively influence the research results and interpretation. Since different research methods should be combined for the first time, measuring equipment and research methodology ought to be modified. The measuring devices and equipment should be modified by changing the measuring characteristics of the sensors (adaptation of their sensibilities in the desired frequency spectres and the measuring range), by adapting length of seismic cables to the desired depth of work, and to carry out additional structural solutions and adaptations which will provide optimum working and measuring conditions.

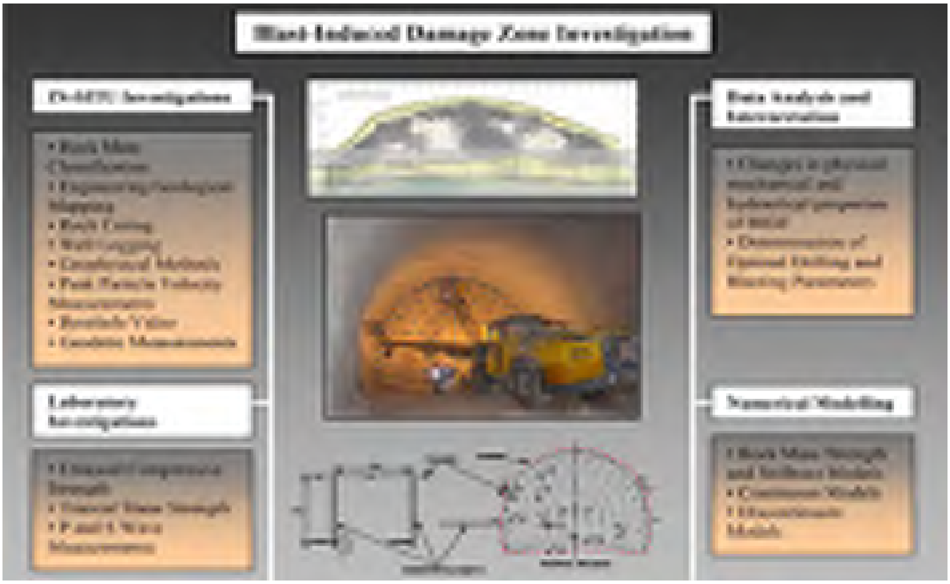

Figure 6 Proposed methodology of BIDZ research

The goals of proposed research methodology are:
- to obtain a more realistic picture of the medium complexity in order to detect BIDZ;
- to evaluate efficiency of various geophysical methods during the tunnel excavation;
- to compare results from standard and modified research methods;
- to establish and recommend the new methodology of in situ BIDZ research suited to underground space and all its limitations.

References

[1] Miščević, P.: Introduction to engineering rock mechanics (in Croatian), University of Split, Faculty of civil engineering and architecture, 2004.

[2] Hudson, J. and Harrison, J.: Engineering Rock Mechanics, Elsevier Science Ltd., 1997.

[3] Lunardi, P.: Design and Construction of Tunnels, Springer Berlin, 2008.

[4] Antičević, H.: Determination of Blast-Induced Rock Mass Damaged Zone in Tunnels, PhD thesis, University of Zagreb, 2011.

[5] Kujundžić, B., Jovanović, L. & Radosavljević, Z.: Solution du revêtement d'une galerie en charge par applicationd' injections à haute pression, Proc. of 2nd ISRM Congress, Belgrade, 1970.

[6] Kujundžić, B.: Use of tests and monitoring in the design and construction of rock structures, Proc. of 4th Int. Congr. Rock Mech. (ISRM), Rotterdam, pp. 181–186, 1979.

[7] Saiang, D.: Blast-induced damage, Lulea University of Technology, Sweden, pp. 13-27, 2008.

[8] Capozza, F.: Panel discussion: dynamic testing, Int. Symp. on The Geotechnics of Structurally ComplexFormations, Capri, pp. 275–279, 1979.

[9] Malmgren, L., Saiang, D., Töyrä, J. & Bodare, A.: The excavation disturbed zone (EDZ) at Kiirunavaara mine, Sweden – by seismic measurements, Journal of Applied Geophysics 61, pp. 1-15, 2007.

[10] Sjöberg, C.: Cracking zones around slender borehole charges, Proceedings Annual Discussion Meeting BK-79, Swedish Rock Construction Committee, Stockholm, pp. 53-98., 1979.

[11] Sjöberg, C., Larsson, B., Lindrstöm, M. & Palmqvist, K.: A blasting method for controlled crack extension and safety underground, ASF project NO 77/224, Gotheburg, 1977.

[12] Persson, P. and Holmberg, R.: Rock Blasting and Explosives Engineering, CRC Press LLC, 1994.

[13] Chakraborty, A. K., Raina, A. K., Ramulu, M., Jethwa, J. L. & Gupta, R. N.: Lake Tap at Koyna, World Tunneling, pp. 456-460, 1998.

[14] McKown, A. F.: Perimeter controlled blasting for underground excavation in fractured and weathered rocks, Bull. Assoc. Engg. Geol., pp. 461-478, 1986.

[15] Yu, T. and Vongpaisal, S.: New blast damage criteria for underground blasting, CIM Bulletin, vol. 89, pp. 139-145, 1996.

[16] Fjellborg, S. & Olsson, M.: Long drift rounds with large cut holes at LKAB, SveBoFo Report 27, Stockholm, 1997.

[17] Nyberg, U., Fjellborg, S., Olsson, M. & Ouchterlony, F.: Vibration measurements, damage prediction and crack mapping in magnetite ore and waste rock, SveBeFo Report 50 Stockholm, pp. 23-32, 2000.

MONITORING AND SUPERVISION OF TUNNELS IN CROATIA

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Abstract

One of the most responsible and most challenging construction tasks are tunnel excavation and stabilization. Tunnel excavation and support system were developed back in the nineteenth century and significant advances in technology were achieved in the second half of the twentieth century. In order to reduce the risk safety management systems were established. Such system also includes the monitoring, apart from the design requirements on excavation and support and the assessment of stability during construction. Monitoring has the purpose frequently observation and adjustment of excavation and support system to actual soil conditions. Unfortunately, all measures of monitoring are defined only for the tunnel construction period. In Croatia, there are no technical standards for tunnel monitoring during the exploatation. The aim of this paper is to point out the importance of methodology and monitoring procedures that would detect changes in time and ensure the tunnel sustainability. Tunnel monitoring implies a set of operations that can promptly register the events and conditions that could affect the stability, traffic safety and tunnel durability.

Keywords: tunnel, monitoring, tunnel sustainability, traffic safety

1 Introduction

Tunnel excavation and stabilization immediately after excavation is one of the most responsible and most challenging engineering tasks. Especially when dealing with complex engineering geology conditions [1] and tunnels with and large cross-section [2]. New excavation support system was designed by Austrian engineers in fifties. Today it is known as “New Austrian tunnelling method” or NATM and it applies in Croatia. NATM introduced obligatory strain measurement of the tunnel opening (“convergence measurement”), as the basic element of support verification. In Croatia the interactive design method applies in tunnel design [3]. Development and application of interactive geotechnical tunnel design started in Croatia in 1980. [4]. This methodology [5] integrates the empirical, rational and observational approach in the geotechnical design of the tunnel. The observation approach is the second phase of tunnel design defined in main design and carried out during the excavation of the tunnel. Verification of the tunnel support system and modification during the construction phase was done using the actual data obtained from the measurement and the monitoring. Geotechnical measurements combined with back analysis are basic part of interactive design concept [6]. Tunnel safety is ensured in the design at the beginning and continues with traffic regulation, tunnel installation check-up like; ventilation, fire protection, lighting and sewage. Continuous supervision or monitoring during exploitation is essential for assessing tunnel safety at any time.

2 Importance of tunnel monitoring

Geotechnical designing in Croatia is conducted in accordance with Eurocode 7, HRN EN 1997-1 [7]. This standard is dealing with the geotechnical design in order to ensure the construction strength, stability, serviceability and durability. This standard envisages the elaboration of the design complexity and monitoring in three geotechnical categories. Minimum geotechnical requirements for each category are established based on the construction complexity, ground conditions and risk. Tunnels in general are classified in category 3. Exceptions are the tunnels in a solid rock classified in category 2. Monitoring programme is defined depending on the geotechnical category. For example, for geotechnical category 1 monitoring programme can be limited to inspection, simple quality controls and qualitative assessment of the structure behaviour. For geotechnical category 2, measurements of ground properties and the structures behaviour sholud often be required. And for geotechnical category 3, additional measurements are required with complex analysis and interpretation of the measured values. Figure 1. shows a crack in secondary concrete tunnel lining detected during exploataion. This crack is an example of tunnel condition which should be monitored and measured, because it directly affect the tunnel safety.

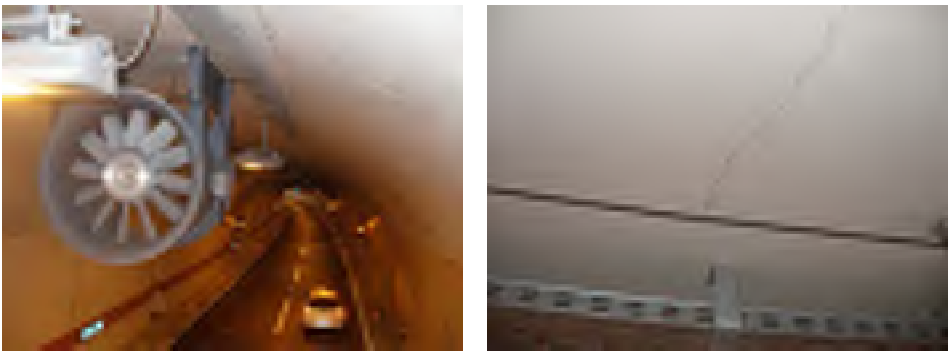

Figure 1 Crack in secondary concrete tunnel lining

Tunnel monitoring and maintenance according to standard [7] is process defined in the geotechnical design. Geotechnical design should clearly determine what to monitor during the construction and maintenance during the exploatation (HRN EN 1997-1: 2.8.(4)P). Matešić and Mihaljević [8] gave guidelines for minimum requirements to define monitoring equipment, number and position before construction and to anticipate the costs for equipment installation and measurements.
As well as the technical monitoring can promptly identify and register all the events and conditions that could impact the safety and surrounding area for what the technical monitoring is really important is the rational maintenance. Remediation of these structures is very expensive and it is important to identify and remove any damage in time.
Maintenance costs and security and construction usability are strongly connected. In order to achieve a constant level of construction security the maintenance costs are necessary. Figure 3. is showing the connection between the cost of preventive maintenance (line 1 ‘), which are necessary to continuously maintain the designed security level (line 1). When maintenance is irregular the security level is constantly decreasing (line 2) and by activation of corrective maintenance (line 2 ‘) at a critical point, the costs are higher than the preventive maintenance in order to restore and maintain the construction security.
On the Chekka tunnel experience the importance of even minimal technical monitoring is shown. Sometimes the forced abandonment of important infrastructure funds for the structures maintaining can lead to serious damage that are too expensive to repair [9], and the

impact of structures on the surface or in the vicinity of the tunnel is given in the example of removal of a building of 14 floors which caused the unloading of soil and underground structures. Due to the unloading of the soil underground had lifted for about 2 cm [10].

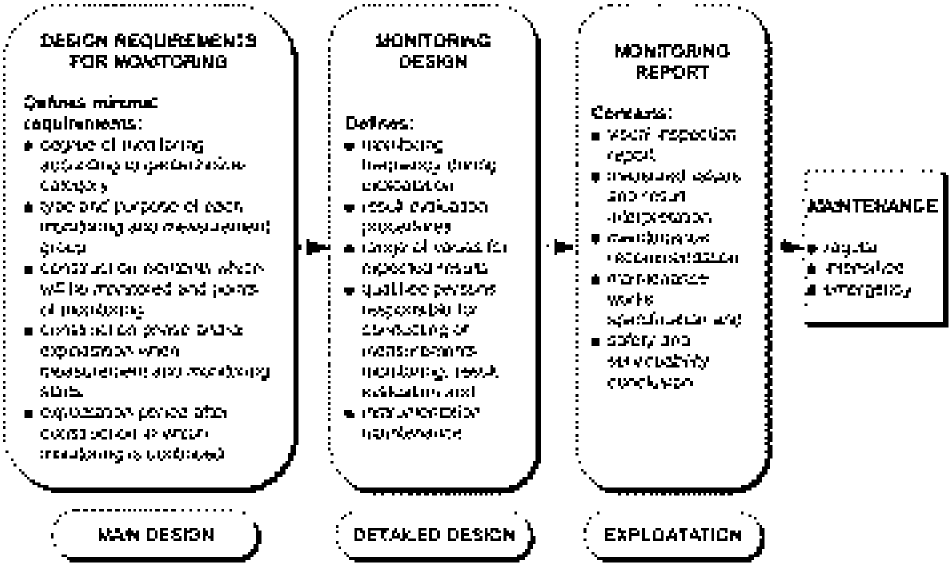

Figure 2 Monitoring and maintenance flowchart [8]

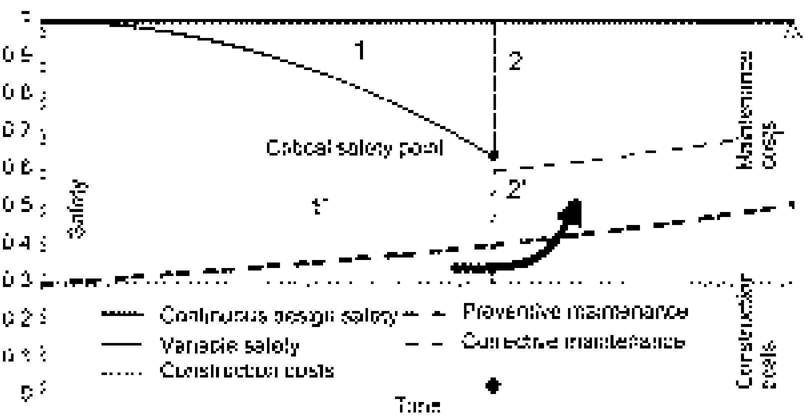

Figure 3 Time related connection between maintenance cost and safety level

3 Guidelines for the technical monitoring of tunnels

According to EN 1997-1 [7] in terms of supervision and behaviour monitoring, geotechnical design includes: the purpose of each group of observations and measurements, parts of the structure whose behaviour will be monitored and places that will be observed, the frequency of measurements, a way of evaluation results, the range of values within the results are expected, the period in which monitoring needs to be continued after the end of construction and the persons responsible for the implementation of measurement and observation, interpretation of results and maintenance of instruments. In order to obtain an overview of the tunnel behaviour and the behaviour of the tunnel sections a full range of monitoring should be applied. These monitoring includes; technical, seismological and hydrogeological observation. The part of the technical observations are visual observations which includes observations of the damage resulting from accident, temperature changes, fire, earthquakes, tunnel settlement or settlement out of the tunnel area. Cracks in concrete lining, bulges or depressions in the tunnel, landslides, slopes at the entrance of the tunnel or at the area of the tunnel, seepage or wet surface in concrete lining and condition of concrete surfaces are observed and monitored. Apart from visual observations, technical monitoring techniques are well advanced and uses devices for 3D measurements, extensometers, measuring anchors, pressure cells, deformeters, piezometers, magnetic extensometers, sliding micrometers and other field and laboratory tests. Monitoring frequency are different according to state standards. For tunnels and subways periods and scope of observation are defined in DIN 1076, BOStrab and RIL 853 [11]. DIN 1076 [12] includes the Main inspection, every 6 years, of all relevant objects like foundations, loadings, traffic signs, structural elements, corrosion etc. The standard inspection, every 3 years in order to review the relevant elements from the Main inspection and supervision, twice per year, which means visual observation to gather the information about the visual irregularities. The Special inspection is recommended after the incidents such as fire, earthquake or water breakthrough. Guidelines for the frequency of engineering inspection are given in the flowchart Figure 4. The results of the monitoring documentation are detected defects which are than classified into the three categories. ''Urgent'', which includes immediately repairs, then ''not urgent'' which requires repair in the medium term and the category ''further observation that involves repairs in the long term and the category of the ''further monitoring'' that includes the repairs in the long term. The ultimate success of monitoring depends of how regular, systematic, arrange and transparent the technical documentation is. Documentation of monitoring should be arranged and conduct in way that it allows the constant and complete insight into the observed tunnel condition at any given time. That means a set of all documents which are showing the actual constructed state of tunnel and whole technical monitoring net as well as a set of the documents during the life of the tunnel showing the state which is established by a systematic technical observations.

Table 1 Engineering inspection and supervision [11]

DIN 1076	DB Netz AG RIL 853	BOStrab
Supervision (2x a year)	Supervision (4x a year)	
Standard inspection (every 3 years)	Examination (if required)	
Main inspection (every 6 years)	Assessment (every 3 or 6 years)	Inspection (every 10 years)
Special inspection (if required)	Special inspection (if required)	

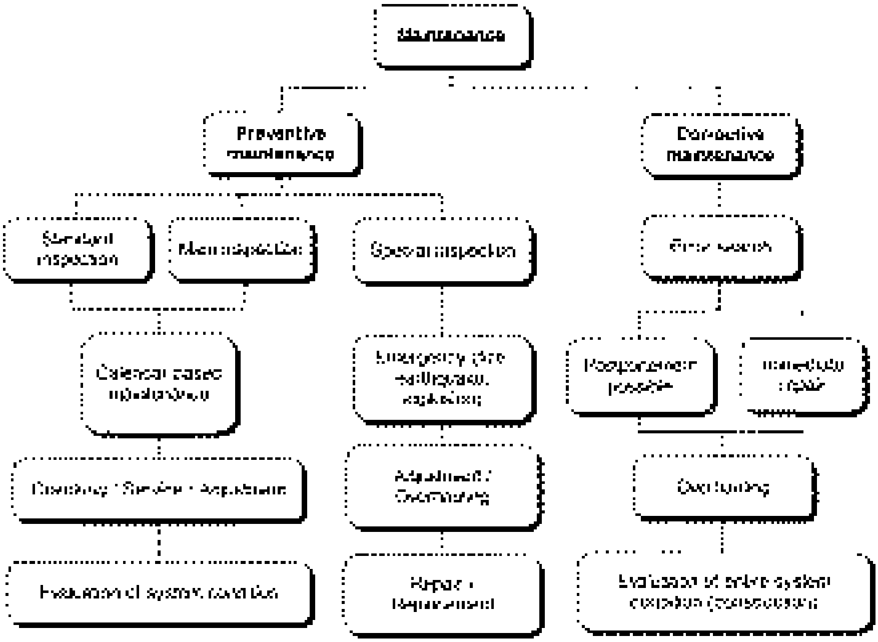

Figure 4 Time related connection between maintenance cost and safety level

4 Concusion

The main reason for the technical monitoring of tunnel is primarily for the public safety. It has the purpose to determine whether the construction behavior is normal, or whether there is a deformation or displacement, which could be a sign of structure disturbance, foundation, or nearby area. The other reason for the monitoring is because of rational maintenance. Remediation of these constructions is very delicate and expensive. Therefore there is a need to take any damage and undesirable phenomena detected as soon as possible and solved until any major proportions.

Besides technical monitoring of constructions in general and the tunnel as well is expected by the Croatian standard EN 1997-1 and the National Annex BS EN 1997-1:2008 / NA.

Since there are no national standards or regulations in Croatia this paper provides the guidelines for the technical monitoring by emphasizing the importance of using the same, and emphasizes the need to develop the regulations for technical observation of the tunnels.

References

[1] Garašić, M., & Ašperger, M.: The engineering geological conditions in the tunnel Papratno on the peninsula Pelješac, Proceedings of the 1th Yugoslavian Symposium of the tunnels, Brioni, pp. 77-82, 1988 (in Croatian).

[2] Jašarević, I., Hudec, M., Stojković, B., Plamenac, M., Vrkljan, I & Maremče, M.: Tunnels with large cross – sections HE Bekhme – excavation ensurance, Proceedings of the 1th Yugoslavian Symposium of the tunnels, Brioni, pp. 165,1988. (in Croatian).

[3] Stojković, B.: Road tunnel design in rock mass using integral method, Ms. Thesis. Zagreb, University of Zagreb (in Croatian), Faculty of Civil Engineering, 1986.

[4] Bušelić, Lj.: Road tunnels in SR Croatia, Proceedings of the 1th Yugoslavian Symposium of the tunnels, Brioni, pp. 221-228, 1988 (in Croatian).

[5] Stojković, B., Stanić, B. & Kovačević, M. S.: Methodology of geotechnical design of Sveti Marko tunnel. Građevinar, 53(8), pp. 507-516, 2001 (in Croatian).

[6] Kovačević, M.S.: The observational Method and the Use of Geotechnical Measurements, In Proc. 13th Europ. Conf. on Soil Mech. and Geotech. Eng. Prague. pp. 575-582, 2003.

[7] EN 1997-1:2004 " Eurokod 7: Geotechnical design (Eurocode 7: Geotehnical design – Part 1: General rules)

[8] Matešić, L., Mihaljević, I.: Monitoring of structures after construction in accordance with Eurocode 7, Proceedings of the 6th Croatian geotechnical Society, Remediation Monitoring and maintenance in Geotechnics, Zadar, pp. 54-56, 2013, (in Croatian).

[9] Sadek, S., Hamad, B.: Lack of Maintenance Compromises Tunnel Structural Safety, International Journal of Geoengineering Case Histories, Vol. 1, Issue 2, pp. 136-153.

[10] Katzenbach, R., Leppla, S.: Deformation behaviour of clay due to unloading and the consequences on construction projects in inner cities, Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, pp. 2023-2026, 2013.

[11] Katzenbach, R., Leppla, S.: Engineering inspection and supervision of tunnels and underground stations of urban metro systems, Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, pp. 1727-1730, 2013.

[12] Deutsches Institut fur Normung e.V. 1999. DIN 1076: Ingenieurbauwerke im Zuge von Straßen und Wegen. Beuth Verlag, Berlin.

SV. ILIJA TUNNELS THROUGH BIOKOVO MOUNTAIN

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Abstract

The Sv. Ilija tunnel, 4.1 km long with longitudinal alignment gradient of 2.5%, was designed and constructed to improve transport links of the Makarska Riviera with the rest of the roads in Croatia. This traffic connection between the Makarska seaside and the A1 highway: Zagreb-Split-Ploče will carry up to 8600 vehicles/day as compared to the current number of 3000 vehicles per day (estimates for the year 2026). The longitudinal ventilation system with adequate improvement was installed by building service tunnel connected to the main tunnel tube every 700 m for fire trucks and every 250 to 300 meter distance for pedestrian crosswalks. The main building (geotechnical) project was developed by SMAGRA Ltd. Zagreb. According to the Traffic study (IGH, 1999) expected construction costs amount to 428.000.000 HRK. The estimated construction cost value based on the main construction project is as follows: a) main tunnel – 320.728.00 HRK and b) service tunnel – 64.145.000 HRK (7,5 HRK = 1,0 EUR). The main construction project for the building permit consists of eight books Phase I and thirteen books Phase II (installation projects). The building permit for the tunnel construction was issued on February 19, 2001. The construction of the tunnel was completed and handed over to the operator for use in 2013.

Keywords: tunnel, main construction project, geotechnical project, engineering geology, rock mass

1 Introduction

The Sv. Ilija tunnel, 4100 m in length and with the longitudinal alignment gradient of 2.5% connecting the Makarska coastline with the A1 motorway, was designed and constructed through the Biokovo rocky mountain in the direction of Baško Polje. Considering the complex engineering geology situation and high seismicity level (IX seismic zone), the construction process had to deal with the complex structure of the rocky massif, partially defined underground water level and changing geotechnical parameters (compressive strength, deformability module). The height of the overburden rock mass is about 1300 m, so far the highest overlaying rock mass in tunnel construction in the Republic of Croatia. Considering the length of the tunnel, it should be notified that the tunnel was dimensioned to comply with the traffic forecast of 8600 vehicles/day. By the European Tunnel Assessment Programme classification (Annex 4/2004) the tunnel is assigned to current updated Category II list. As for the traffic, the ventilation must ensure that the emission of the fuel gas is within the permissible limits. The European Union regulations (Annex 4/2004) require adequate equipment (lightning, traffic signaling, hydration system, fire alarm system) with no increase in the traffic accident risk. Having in mind the tunnel length of 4100 m, with the threshold limit value for longitudinal ventilation of 4000 m, the longitudinal ventilation system was installed to improve service tunnel connected to the main tunnel. This set up maintains traffic connections for fire trucks every 700 m and walkways for passenger evacuation every 250 to 300 m.

Figure 1 The tunnel route

2 Engineering geology, Hydrogeology, Geophysical and Geotechnical surveys

Submitting project documentation for building permit required engineering geological, hydrological and geophysical surveys as well as geomechanical laboratory sample testing. The respective geotechnical project incorporates synthesized field research findings. Engineering geological and hydrogeological investigations were conducted at the north and south portal of the Sv. Ilija tunnel on the Baška Voda-Zagvozd-Imotski road. Conducted research involved detailed geological mapping, field screening of lithological progression, element layers measurement and identification of structural tectonic elements, such as outspreading and slope of fault zones, measuring of breach systems, breach system penetrability, its stuffing and others. In addition to geological prospection, hydrogeological prospection on tunnel portal locations was also conducted. Restricted hydrogeological investigations were carried out by measuring the hole drilled in the near vicinity of the tunnel´s north portal and hydrogeological categorization of the rocks in the exploration area. Based on geological mapping specific locations for setting geophysical profiles were determined. Accordingly, tectonic systems in the portal zone of the tunnel were confirmed, i.e. geometry zone of the talus deposits determined by the geological mapping on south tunnel´s portal.
Based on geological prospecting the following lithostratigraphic segments were selected:
- well-bedded limestones and dolomites $J_{1,2}$;
- well-bedded limestones $J_{2,3}$;
- limestones and dolomites with massive limestone cartridges $^{1}K_{1}$;
- dolomites, dolomitic limestones and limestones $K_{1,2}$;
- poorly bedded bioaccumulated limestones K_2^{3};
- bulbous glauconitic limestones and marl E_2;
- talus breccias Q_d.

Structural-tectonic pattern of the whole Biokovo area is characterized by a high degree of tectonic disturbance as a result of geological movements from older Mesozoic until today. The structural unit of Biokovo is clearly bordered with two strong dislocations from the southwest and northeast side. In the southwest the Jurassic and Cretaceous limestones are much drawn to the Eocene detrial Makarska units. As to the northeast the upper Cretaceous limestones of the Slivno structural units are partly drawn to the equivalent upper Cretaceous sediments of the Biokovo units.

Hydrogeological relations at the location are determined by structural-tectonic interrelation, relief and hydrogeological properties of the represented lithologic segments. The relief is characterized by steep slopes made up of limestones. The additional relief expression is caused by fault and fissure systems with occasionally occurring vertical sections. At the north portal location heavily karstified limestones with numerous karst formations and open fracture system are recorded. The emergence of caverns and individual large cave objects in this part of the field was anticipated. Following the measured data, the depth of karstification was not approximately 30 m below, except in places where, due to tectonics, deeper chemical limestone abrasion was encouraged. Chemical limestone abrasion (karstification) spreads over the investigation area. Under such conditions, surface humus layer fails to occur. Consequently, the infiltration of rainwater into underground occurs entirely in the absence of superficial outflow. Numerous springs on the coastal area, as well as the total absence of wells and surface runoffs, confirm the appearance of underground storm water runoff.
Locations for profiles of geophysical testing were determined according to field screening and data analysis. Two refraction profiles in the southern portal and three refraction profiles in the northern portal were derived. Rank interpretation of speed amount is up to 6500 m/s, and reached interpretation depth is between 20 and 40 meters. Based on the refractive research results, the grade levels of both tunnel tubes at the northern portal are located in the area of compact carbonate rocks (seismic wave velocity › 4000 m/s) of which only the deviation of the area around chainage 3 + 700 was recorded – the spot where strongly weathered fault zone carbonate rocks were observed. The soil difference between the main and service tunnel tube was not evidenced by the cross section interpretation results. The interpretation of geophysical profiles at the tunnel tube grade level at the south portal revealed the compact carbonate. However, narrow portal sites were rugged with distinct fault zones.

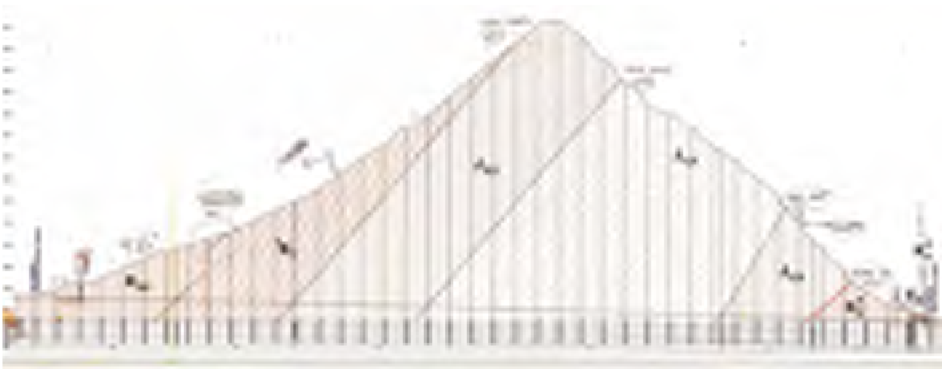

Figure 2 Engineering geological cross section

The 23.8 m deep exploration well was drilled in the hinterland of the north portal. Core drill was determined in the field, RQD and relatively small quantities of groundwater levels measured. After determination of cores for laboratory testing samples were taken to the depth of 8.9 m. Thereafter, laboratory engineering mechanic testing was performed at the Faculty of Civil Engineering, University of Zagreb. The values of compressive strength (σ_{tl}) increase with sampling depth ranging between 62 and 133 MPa, while the values of tensile strength (σ_{vl}) obtained by the Brazilian test range between 7 and 19.5 MPa. Also, elasticity module E of 69 GPa value was determined.
The carried out surveys and collected results were used for designing the major geotechnical project of the main and service tube of the Sv. Ilija tunnel.

Figure 3 The north portal of the main and service tunnel tube

3 Main construction project for obtaining the building permit

The main construction project and project solution are designed for the Sv. Ilija tunnel between north and south portals on the Baška Voda-Zagvozd-Imotski road. Major construction project follows project solutions derived from Expert Elicitation for issuing the location permit. Documents required for obtaining a location permit were prepared by Geoprojekt Company in November 2004. The location permit was issued by County Road Administration. The lack of visibility in the tunnel portal zones is considered to be the basic disadvantage of the demonstrated project solution. The north portal lies within a 500 m radius, and the south portal within 300 m horizontal radius zone. The north portal is 363.70 m above the sea level, and the north portal 263.02 m above sea level with unilateral longitudinal slope of 2.50%. According to the European Agreement (Annex 4/2004) tunnels are categorized into classes. Constructing measures for respective traffic volume (vehicles/day) are specified by the same agreement as follows:
- fire roads;
- emergency exit;
- rescue emergency cross passages every 1500 m;
- escape windows;
- middle lane crossing;
- emergency roadside stop.

As for the groundwater coming from direction of Imotski, the designing of one-side slope of the roadway tunnel was considered necessary according to the conducted research. [Normal] The use of SI units is strongly recommended and mixed units are to be avoided.

3.1 Proposal for amendments to the main construction project

The change in the tunnel layout position was anticipated in the south portal due to the exact location of faults and talus deposit zone. In this view it is important to consider also the high level of seismicity (IX zone). The south portal should be moved closer to settlement, i.e. east of the talus and not towards the quarry. Due to the possible rising of groundwater above the tunnel alignment, additional explorations in the northern portal zone were required prior to executive project finalization.

3.2 Rock mass geotechnical data

Lithostratigraphic units described in previous section and certain geotechnical parameters were singled out on account of geological mapping, exploration drilling in portal zones, geophysical surveys (seismic refraction) on individual longitudinal and transverse profiles and aircraft shot analysis of wider Biokovo area.

Table 1 Adopted uniaxial compressive strength of homogeneous sample rocks σ_{ci}, Hoek-Brown constants m_i and geological strength index GSI for lithostratigraphic units passed by the tunnel

Lithostratigraphic units	$K_{1,2}$	$^{1}K_{1}$	$J_{2,3}$	$J_{1,2}$	$J_{2,3}$	$K_{2,3}$	E_2	K_2^3
Stationing [km]	3+500-4+148	4+148-4+684	4+684-5+299	5+299-6+643	6+643-7+055	7+055-7+506	7+506-7+519	7+519-7+600
σ_{ci} [MPa]	92	118	80	70	80	92	10	60
m_i [l]	10	10	10	10	10	10	10	10
GSI [l]	14	70	60	55	60	54	20	34

Estimated values of GSI (Geological Strength Index) based on the speed of longitudinal waves in the northern and southern sides are expressed by equation (Bienawski, Barton, 1991):

$$GSI = 9\ln Q + 4 \tag{1}$$

$$Q=10[(Vp-3500)/1000] \tag{2}$$

where:

Vp velocity of longitudinal waves;
Q quality of the rock mass classification by NGI classification.

Table 2 Firmness and rock mass deformability parameters on the alignment level of tunnel by lithological units

Lithostratigraphic units	$K_{1,2}$	$^{1}K_{1}$	$J_{2,3}$	$J_{1,2}$	$J_{2,3}$	$K_{2,3}$	E_2	K_2^3
Stationing [km]	3+500-4+148	4+14 -4+684	4+684-5+299	5+299 6+643	6+643 7+055	7+055 7+506	7+506 -7+519	7+519 -7+600
Max. Overlay height (m)	237	460	879	1319	734	361	45	41
Cohesion c' (MPa)	1,933	4,926	4,086	4,439	3,704	2,377	0,1283	0,3884
Friction angle ρ (°)	46,50	47,09	37,40	31,90	36,77	43,51	31,39	49,64
Frimness σ_{cm}(MPa)	9,67	25,29	16,53	15,96	15,44	11,06	0,46	2,11
Def. module Em(MPa))	12075	31622	15905	11157	15905	12075	562	3083

Uniaxial rock mass firmness σ_{cm} from the Mohr Colomb diffraction criteria according to the equation (Hoek, Brown, 2002):

$$\sigma_{cm} = \frac{2c'\cos\varphi}{1-\sin\varphi} \tag{3}$$

where c '(cohesion) and φ (friction angle) are determined from the nonlinear relationship σ_n-τ for values of normal overlay strainings.

Rock mass deformation module in lithostratigraphic units was determined using the equation (Hoek, Brown, 2002):

$$E_m = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \times 10^{[GSI-10]40} \tag{4}$$

where D stands for mass deformation factor in contour surroundings depending on excavation technology.

3.3 Calculation of stability

Calculation of stability is based on the numerical analysis of various forms and dimensions of the tunnel support system (Hoek, Marinos 2000)

Table 3 Primary tunnel support and excavation method following geomechanical classification for horseshoe tunnels ranging up to 10 m

Rock mass category	Excavation	Primary support		
		Passive anchor diameter 20mm	**Shotcrete**	**Steel arches**
RMR=81-100 I Very good rock	Excavation throughout the profile. 3 m excavation step	General support not required, except sporadic single anchoring		
RMR=61-80 II Good rock	Excavation throughout the profile. 1-1.5 m excavation step. Finish support 20 m away from the front.	Sporadic anchoring in the roof. Anchors 3 m length at 25 m distance. Partially steel mesh.	50 mm on roof where necessary	Without arches
RMR=41-60 III Favourable rock	Excavation in two phases. 1.5-3 m step excavation into the roof. Support setting after every single tunnel lining excavation. Finish suppport at 10 m distance from the front.	Systematic bolting anchors 4m length at 1.5- 2 m distance in roof and walls. Steel mesh in roof.	50-150 mm on roof and 30 mm on walls.	Without arches
RMR=21-40 IV Poor rock	Excavation in two phases. 1-1.5 m step excavation into the roof. Support setting along with excavation.	Systemic bolting anchors 4.5 m length at 1-1.5 m distance in roof and walls. Steel mesh in roof and walls.	100-150 mm on roof and 100 mm on walls.	Light to medium spaced 1.5 m where necessary
RMR‹20 V Very poor rock	Elaboration of the excavation profile in the roof 0.5-1.5 m. Support setting along with excavation. Installation of shotcrete immediately after excavation.	Systemic bolting anchors 5-6m length at 1-1.5 m distance in roof and walls. Steel mesh in roof and walls. Undershot anchor roofing.	150-200 mm on roof, 150 mm on walls and 50 on front	Moderate to severe at 0.75 m distance. Steel props roofing if necessary. Closed undershot roofing.

Figure 4 Support systems of the main tunnel – type III

Figure 5 Support systems of the main tunnel – type V

3.4 Geotechnical measurements

During the tunneling convergence measuring on triangle sections (one point in calotte and two in hips) is obligatory. Convergence measurements between points in the cranium and hips should be performed immediately after installing the adequate support system.

4 Conclusion

Standard methods for main and service tunnel design were applied. Considering the insufficient exploration of the rock massif (engineering geological aspect and hydrogeology), potential hazards and increasing risk may appear.
Design and construction of this long and deep tunnel (the largest overlay 1300 m) in the karst area of the Adriatic coast were carried out largely on account of the assessment of geological, engineering geological and hydrological parameters with lots of uncertainties especially in terms of caverns and sudden groundwater penetration.
As to the morphology of the surface, it was impossible to set up the longitudinal geophysical profile, leading to unsatisfactory results. This was confirmed in terms of bridging over large caverns in the service tunnel near the northern portal. For derived refractive profiles, the roadway level was not scored in most parts of the tunnel.
Hydrogeological research studies were not sufficient enough especially in the northern zone of the portal and southern portal of the fault zone. Additional hydrogeological investigation is strongly suggested in the project documentation.
The vertical pressure at the tunnel outbreak on the edge of the rock hole is 70 Mpa, which may increase the risk in case of adverse loads ((hydrostatic pressure and earthquake).
Connecting of drainage pipes, set on the edge of the support system and DN 150 rock mass on to the DN 600 central drainage pipe in the middle of the tunnel, increases the risk of groundwater penetration.
The tunnel construction started in 2008 and finished in 2013. Construction work was performed by Hidroelektra, and Konstruktor Split, companies.
Building permit for Sv. Ilija tunnel was issued on February 19, 2001. Responsible for obtaining necessary regulatory permits are as follows: Cad Com, Zagreb; Geoaqua, Zagreb; Moho, Zagreb; Smagra, Zagreb; Faculty of Civil Engineering University of Zagreb; Promel project, Zagreb; and Dalekovod, Zagreb.

References

[1] Barton, N.: Application of Q-system and index tests estimate shear strength and deformability of rock masses, Proc. Int .Symp. Engineering Geology and Underground constructions, Lisbon, Vol 2.,pp. 51-70,1983.

[2] Barton,N.: Geotechnical design, World Tunnelling Vol.4, pp. 410-416,1991.

[3] Hoek, E. & Marinos, P.: Predicting Tunnel squeezing problems in weak heterogeneous rock masses, Tunnels and Tunelling International, 2000.

[4] Lugomer, D., Bubanović, B., Redžepagić, M.: Sveti Rok Tunnel excavation-north side, Scientific Symposium, Rock Mechanics and Tunneling, Zagreb, pp.199-208, 1999.

[5] Redžepagić, M.: Overbreak in Tuhobić Tunnel project – west side, Scientific Symposium, Rock Mechanics and Tunneling, Zagreb, pp.193-198, 1999.

8 INTEGRATED TIMETABLES ON RAILWAYS

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3rd International Conference on Road and Rail Infrastructure

MICROSCOPIC SIMULATION OF RAILWAY OPERATION FOR DEVELOPING INTEGRATED TIMETABLES

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Abstract

OpenTrack is a user-friendly railroad network simulation program developed at the Swiss Federal Institute of Technology's Institute for Transportation Planning and Systems (ETH IVT). It is a microscopic model that simulates rail system operations based on user defined train, infrastructure, and timetable databases. OpenTrack functions as a railroad laboratory, by, for example, allowing users to define incidents and take infrastructure out of service to evaluate alternative scenarios. The program uses a mixed discrete/continuous simulation process that calculates both the continuous solution of train motion equations and the discrete processes of signal box states and delay distributions. It generates a wide variety data that can be easily presented in many formats including graphs (e.g. time-space diagrams), tables, and images. OpenTrack's main uses have been to evaluate and test infrastructure plans and operating schedules to optimize network and timetable design. It can be run on several different computer platforms and incorporates the benefits of object oriented programming language with a common data interface structure.

Keywords: railway simulation, railway planning, railway operations analysis, RailML, XML, object oriented modelling

1 Introduction

At its most basic level a rail simulation program is a set of simplifying mathematical assumptions that attempt to duplicate actual train operations. It allows users to evaluate the impacts of user-defined changes on the rail system operation. There are many types of railroad simulation programs; each of which is designed to answer a different type of question. For example there are models designed to help dispatchers decide how to route trains through a network in real time. Similarly there are models designed to help planners identify infrastructure requirements for planned schedules ten years in the future. As with most planning processes, a critical component of railroad planning is choosing the best computer simulation tool to use in any given situation. Rail simulation models can be categorized as macroscopic or microscopic similar to other transportation models. Both types are outlined below.

1.1 Macroscopic Simulation Models

Macroscopic models use average or other statistical data to evaluate operation of the transportation system; they do not model individual unit (e.g. train) operations nor do they consider how trains are impacted by other trains or how safety systems impact train performance. A common type of macroscopic model is the NEMO model developed at the University of Hannover.

1.2 Microscopic Simulation Models

Microscopic simulation models attempt to replicate the actual operation of a railroad over time. They do this by modeling the operation of each individual train during a user-defined time step (often one second) and then repeating the process for the entire simulation period. Microscopic models consider the impact of trains on each other when they simulate train operations. For example, if during a particular time step, a train occupies a block that a second train wishes to occupy, then the program will model the second train as performing according to the (user defined) safety system parameters. In the case of a simple block signal system the second train would stop at the block entry signal and wait until the first train clears the block before proceeding. There are two types of microscopic models: synchronous and asynchronous. Synchronous models simulate all train operations in a single model run while asynchronous models simulate operations in a series of model runs.
In an asynchronous simulation the highest priority trains are modeled in the first run, then this schedule is locked and the second set of trains is modeled; the operation of the second set of trains does not impact the first set of trains, but is impacted by the first set (similarly, the operation of the third set of trains does not impact the first or second sets, and so forth). These types of models are often used for timetable construction since they can replicate ideal operational planning. A good example of this type of program is STRESI developed by RWTH in Aachen.
In contrast, synchronous models simulate all the trains operating in the modeled network at the same time, thus they provide a good way to simulate realistic operating situations (e.g. the impact of train delays on network operations). These models enable users to specify rules that the program uses to make dispatching decisions when there are conflicts between trains. These rules include simple train priorities (e.g. passenger trains before freight trains) as well as more complex rules designed to optimize some particular function (e.g. dynamic overtaking). In essence these models attempt to replicate good dispatching decisions.
OpenTrack is a microscopic synchronous railroad simulation model. It provides users with a great deal of flexibility for defining different dispatching logic as well as operational variables in a user-friendly manner.

2 OpenTrack Rail Simulation Program

OpenTrack was developed at the Swiss Federal Institute of Technology's Institute for Transportation Planning and Systems (ETH IVT). The project's goal was development of a user-friendly railroad simulation program that can run on different computer platforms and can answer many different questions about railway operations. Figure 1 illustrates the three main elements of OpenTrack: data input, simulation, and output.
OpenTrack is a microscopic synchronous railroad simulation model. As such it simulates the behaviour of all railway elements (infrastructure network, rolling stock, and timetable) as well as all the processes between them. It can be easily used for many different types of projects including testing the stability of a new timetable, evaluating the benefits of different long-term infrastructure improvement programs, and analyzing the impacts of different rolling stock.

2.1 Input Data

OpenTrack administers input data in three modules: rolling stock (trains), infrastructure, and timetable. Users enter input information into these modules and OpenTrack stores it in a database structure. Once data has been entered into the program, it can be used in many different simulation projects. For example, once a certain locomotive type has been entered into the database, that locomotive can be used in any simulation performed with OpenTrack. Similarly, different segments of the infrastructure network can be entered separately into the database and then used individually to model operations on the particular segment or together to model larger networks.

Figure 1 Data flow during a simulation project

Train data (locomotive and wagons) is entered into the OpenTrack database with easy to use forms displayed using pull down menus. Infrastructure data (e.g. track layout, signal type/location) is entered with a user-friendly graphical interface; quantitative infrastructure data (e.g. elevation) is added using input forms linked to the graphical elements. Following completion of the RailML data structure for rolling stock and infrastructure, OpenTrack will be modified to enable train and infrastructure data to be directly imported from RailML data files. Timetable data is entered into the OpenTrack database using forms. These forms include shortcuts that enable data input to be completed efficiently. For example, users can designate hourly trains that follow the same station stopping pattern an hour later. Since OpenTrack uses the RailML structure for timetable data, timetable data can also be entered directly from various different program output files as well as database files.
One advantage of OpenTrack is that it enables users to adjust many variables that impact railroad operations. For example, users can simulate the impact of weather on traction by specifying the adhesion scenario (good, normal, bad). OpenTrack then estimates locomotive traction power using a percentage (also user-defined) of that calculated using the Curtius and Kniffler formula. While OpenTrack provides standard default values for all variables, having the ability to adjust variables makes the program quite useful.

2.2 OpenTrack Simulation Process

In order to run a simulation using OpenTrack the user specifies the trains, infrastructure and timetable to be modeled along with a series of simulation parameters (e.g. animation formats) on a preferences window. During the simulation, OpenTrack attempts to meet the user-defined timetable on the specified infrastructure network based on the train characteristics. OpenTrack uses a mixed continuous/discrete simulation process that allows a time driven running of all the continuous and discrete processes (of both the vehicles and the safety systems) under the conditions of the integrated dispatching rules.
The continuous simulation is dynamic calculation of train movements based on Newton's motion formulas. For each time step, the maximum force between the locomotive's wheels and the tracks is calculated and then used to calculate acceleration. Next, the acceleration function is integrated to provide the train's speed function and is integrated a second time to provide the train's position function.

The discrete simulation process models operation of the safety systems; in other words, train movements are governed by the track network's signals. Therefore, parameters including occupied track sections, signal switching times, and restrictive signal states all influence the train performance. OpenTrack supports traditional multi-aspect signaling systems as well as new moving block train control systems (e.g. European Train Control System – ETCS signaling).

2.3 Dynamic Rail Simulation

OpenTrack is a dynamic rail simulation program. As such, the simulated operation of trains depends on the state of the system at each step in the process as well as the original user-defined objective data (e.g. desired schedule).
A simple way of describing dynamic rail simulation is that the program decides what routes trains use while the program is running. For example, when building the network, users identify various different routes that trains can use between two points; OpenTrack decides, during the simulation, which route the train will use by assigning the train the highest priority route available. If the first priority is not available, OpenTrack will assign the train the second highest priority route and so on.
OpenTrack's dynamic nature allows users to assign certain attributes to specified times in the simulation. Thus, users can assign a delay to a particular train at a given station and time, rather than being limited to assigning a delay at the start and using it through the entire simulation. Similarly, users can define other types of incidents (e.g. infrastructure failures, rolling stock breakdowns) for particular times and places.
Finally, dynamic simulation enables users to run OpenTrack in a step-by-step process and monitor results at each step. Users can also specify exactly what results are displayed on the screen. Running OpenTrack in a step-by-step mode with real time data presented on screen helps users to identify problems and develop alternative solutions.

2.4 OpenTrack Output

One of the major benefits of using an object oriented language is the great variety of data types, presentation formats, and specifications that are available to the user. During the OpenTrack simulation each train feeds a virtual tachograph (output database), which stores data such as acceleration, speed, and distance covered. Storing the data in this way allows users to perform various different evaluations after the simulation has been completed.
OpenTrack allows users to present output data in many different formats including various forms of graphs (e.g. time-space diagrams), tables, and images. Similarly, users can choose to model the entire network or selected parts, depending on their needs. Output can be used either to document a particular simulation scenario or as an interim product designed to help users identify input modifications for another model run.

3 Application of OpenTrack at NeusiedlerSeeBahn (NSB)

The NeusiedlerSeeBahn is responsible for the line from Neusiedl am See to Pamhagen. At Neusiedl am See the branch line from/to Eisenstadt is located. Pamhagen is the last station in Austria before the Hungarian border. From Neusiedl am See a line leads to Parndorf Ort where there is the connection to the Eastern line of Austrian Railways which is connecting Vienna and Budapest. People living at the villages located between Neusiedl am See and Pamhagen typically go for work to Vienna every single day of the week. Therefore the NSB is interested in offering shorter traveling times to Vienna. To achieve this goal some investments were done in the last years to increase the track speed. OpenTrack has been successfully used to evaluate the shortenings of running times for local trains and regional trains.

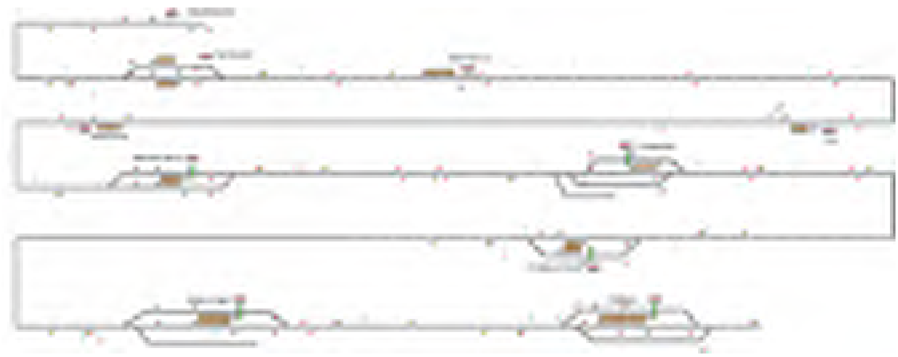

Figure 2 Infrastructure layout of the line from Neusiedl am See to Pamhagen

Figure 2 shows the topology of the line from Neusiedl am See to Pamhagen. Crossing opportunities are located at Bad Neusiedl, Mönchhof-Halbturn, Frauenkirchen, St. Andrä am Zicksee, Wallern and Pamhagen. The first step in the project was the check of the model with the existing timetable. This step allowed the exact calculation of running times reserves included in the existing timetable.

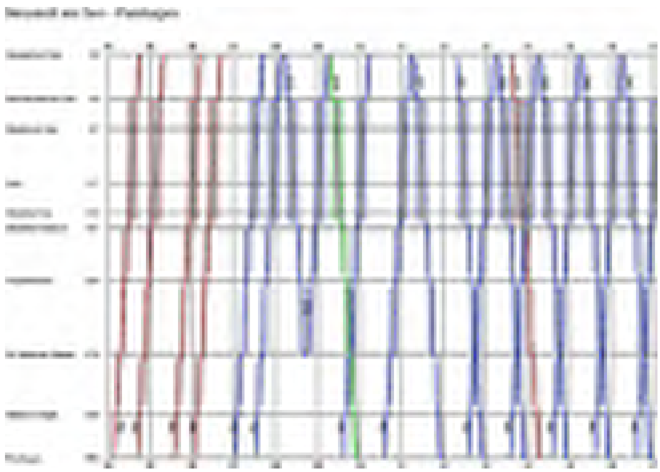

Figure 3 Simulation result of the existing timetable

Figure 3 shows the result from the simulation of the existing timetable. It has to be noted that one course during the day is only going to St. Andrä am Zicksee and not to the terminal station at Pamhagen. Additionally there is no service during the morning hours towards the region between Neusiedl am See and Pamhagen. Exactly the same happens during the evening when there is no service towards Vienna. The reason for this asymmetry in the timetable can be only explained by the saving of operational costs. For pupils there are two additional services during the day to allow them traveling to and back from school. Unfortunately the infrastructure does not allow a crossing between Bad Neusiedl and Mönchhof-Halbturn which would be required to run a 30 minutes interval in both directions. Furthermore the infrastructure model had to be extended with the upgraded track speed limits which had been indicated by markers at the related vertices. Due to the increase of the track speed the crossing shifts from St. Andrä am Zicksee towards Wallern because of keeping the arrival and departure times at Neusiedl am See (see Figure 4).

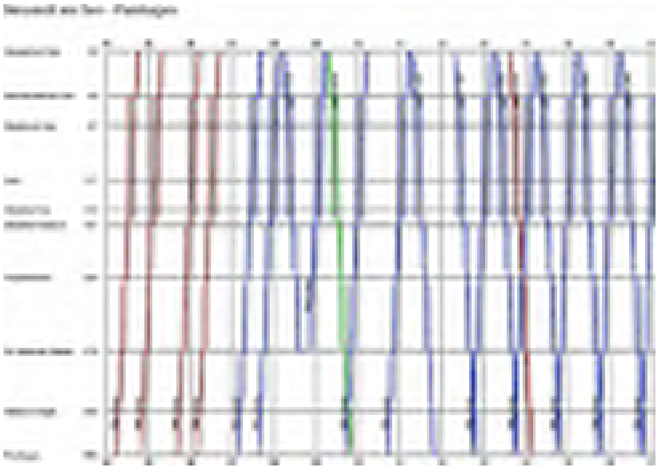

Figure 4 Result of the simulation for upgraded infrastructure

Local and regional trains benefit from the increase of track speed by a shortening of running times of 5 minutes each. Local trains which stop at every single station run in 33 instead of 38 minutes and regional trains run in 28 instead of 33 minutes. A shortening of 5 minutes in terms of running time could be also achieved by introducing today regional trains. Of course the increase of the track speed in combination with the introduction of regional trains leads to an overall reduction of running time of 10 minutes for regional trains. An open point for further investigations is the integration of both services at the same time because each crossing of two trains will lead to an increase of running time while one train has to take the siding track in a crossing station. This could require the demand to upgrade also the frequently used switches to allow higher speeds for the siding track.

4 Conclusions

OpenTrack is an efficient and effective railroad simulation program. It has been successfully used in many different railway planning projects throughout the world. The program's use of object oriented programming and the RailML data structure makes it particularly effective since the program can be modified relatively easily to address specific applications and since data can be transferred easily to and from other programs based on RailML.

References

[1] Huerlimann, D.: Object oriented modeling in railways; ETH Dissertation Nr. 14281; 2001 (in German).

[2] Huerlimann, D., Nash, A.: OpenTrack – Simulation of Railway Networks, User Manual Version 1.6; OpenTrack Railway Technology GmbH; Zürich, Switzerland; 2010; 160 pages.

[3] For more information on OpenTrack see the project website: www.opentrack.ch

A METAHEURISTIC APPROACH FOR INTEGRATED TIMETABLE BASED DESIGN OF RAILWAY INFRASTRUCTURE

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Abstract

The design of new railway infrastructure is a complex planning process in most countries today due to a multitude of requirements. From an operational point of view new infrastructure basically has to fulfill the needs defined by customers. To this end passenger traffic is often organized in an integrated timetable with well defined arrival and departure times at major hub stations. So far there is no automated tool available to help in determining a minimum cost infrastructure fulfilling all the requirements defined by a timetable and the operation of the railway system. Instead, this task is typically carried out manually, based on graphical design, human experience, and also intuition. In our work we model this planning task as a combinatorial network optimization problem, capturing the most essential aspects. We then present a constructive heuristic algorithm that makes use of a dynamic programming procedure for realizing individual commercial stops. Computational experiments on instances derived from real scenarios indicate that the suggested approach is promising and the analysis of obtained results gives useful hints for future work in this area.

Keywords: railway infrastructure design, integrated timetables, combinatorial optimization, dynamic programming, heuristics

1 Introduction

The design of new railway infrastructure is nowadays strongly guided by pre-specified integrated timetables that have been derived from expected traffic to be served [2]. Integrated timetables synchronize the traffic in major nodes (hubs, e.g., main railway stations in major cities) at regular time intervals, ensure connectivity between different lines with minimum waiting times and allow passengers to remember easily the regular departure and arrival times. In many European countries integrated timetables have been successfully introduced in the last years and could prove their substantial advantages.
Implementing the concept of integrated timetables, however, imposes major challenges and constraints, see e.g. [1]. In fact, the almost simultaneous arrival of the most relevant trains at a station and the strongly regulated travel times between stations, which must be multiples of a basic cycle interval, frequently demand extensions of existing railway infrastructure.
So far there is no systematic, automated tool available to aid the design of minimum cost infrastructure that fulfills all the requirements defined by the timetable and the operation of the railway system. Instead, this task is typically carried out manually based on graphical design, human experience, and also intuition, see e.g. [3]. In this paper we present a concrete

combinatorial approach for modeling the basic problem. It considers already existing railway infrastructure as well as various extension possibilities in a fine-grained way. We then suggest a constructive heuristic algorithm for approximately solving this problem, which makes use of a dynamic programming procedure for locally optimal realizing individual commercial stops. The following section presents the formal optimization model, which is based on the model we already introduced in [4] but refined in several details. Our solution method is described in Section 3. Section 4 summarizes experimental results obtained on some benchmark instances that were derived from real scenarios in Austria and have been validated by simulation of railway operation, e.g. OpenTrack or RailSys. Finally, Section 5 concludes the paper and provides thoughts on future work.

2 Combinatorial optimization model

We define the Integrated Timetable Based Design of Railway Infrastructure (TTBDRI) as a combinatorial optimization problem, trying to consider the most relevant real-world aspects. We are given the following input data. An undirected graph G=(V,E) represents the existing railway infrastructure plus all possible extensions on a detailed level. The node set V contains different types of nodes, first of all the following infrastructure nodes corresponding to real objects:

- track segment nodes representing physical, simple track segments of a certain length, they always have at most degree two;
- signal position nodes representing signaling stations; they again always have degree two;
- crossing nodes representing crossings of two lines; their degree always is four;
- switch nodes representing classical switches; they have degree three (or possibly higher if more complex switches are modeled by single nodes).

To model mutually exclusive alternatives for infrastructure extensions, we further use alternative nodes, which have degree k+1 for k mutually exclusive options. Edges E represent the corresponding connections of the respective nodes. Multiple parallel tracks are always modeled by multiple paths. In order to avoid parallel edges and thus the need of a multigraph, it might occasionally be necessary to include virtual nodes; they always have degree two and might be considered as track segment nodes of length zero, i.e., they are just connecting two adjacent objects. Figure 1 shows an example of infrastructure modeling.

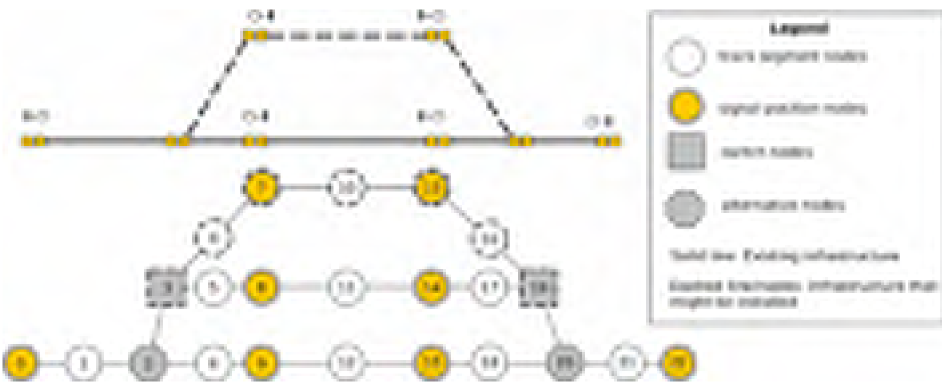

Figure 1 Example for a graph G modeling the existing infrastructure and possible extensions

- Let $R \subseteq V$ be the set of signal position nodes. Paths starting and ending at such nodes and otherwise containing only nodes from $V \setminus R$ are called (compound) routes (“Fahrstraßen”).

Once a compound route is reserved for a train, no other train is allowed to enter any part of this route before the train has left and the route is released again.

- Let the subgraph $G^0=(V^0,E^0)$, with $V^0 \subseteq V$ and $E^0 \subset E$, correspond to the already existing infrastructure and the graph $G'=(V',E')$ with $V'=V \setminus V^0$, $E'=E \setminus E^0$ represent the additionally possible infrastructure by which the existing infrastructure may be extended. Alternative nodes are considered to be part of V^0 if one of the modeled options corresponds to an existing infrastructure, virtual nodes are part of V^0 if both adjacent nodes are also in V^0. All nodes $v \in V$ have associated costs $c_v \geq 0$ and lengths $l_v \geq 0$ with $c_v=0$ for alternative nodes, virtual nodes, and all nodes in $v \in V^0$,$l_v=0$ for signal position nodes, alternative nodes and virtual nodes.
- Set S represents the major railway stations considered in the integrated timetable. Each railway station $s \in S$ has associated a set of simple track segment nodes $V(s) \subset V$ corresponding to the tracks at platforms for boarding/disembarking trains in station s.
- Let $G^D=(V,A)$ be the directed version of graph G, where we have for each edge $(u,v) \in E$ two corresponding oppositely directed arcs $(u,v),(v,u) \in A$.
- An integrated timetable specifies a set of commercial stops $C=\{C_1,\dots,C_{|C|}\}$ to be realized, where a commercial stop $C_l \in C$ is a tuple $(s_l^{start}, s_l^{end}, T_l^{start}, T_l^{end}, G_l^D, train_l, l_l)$ with $s_l^{start}, s_l^{end} \in S$ being start and destination stations and T_l^{start} and T_l^{end} the times when the train may leave station s_l^{start} and has to arrive at station slend latest, respectively. The commercial stop has to be realized by a path in a given subgraph $G_l^D=(V_l,A_l)$ with $V_l \subseteq V$ and $A_l \subseteq A$. It can safely be assumed that G_l^D is acyclic. Finally, $train_l$ indicates the used train's ID. Typically, a train is used for a series of commercial stops. Let $l(train_l)$ refer to the train's length.
- Values $maxspeed_{l,v} l \geq 0$ indicate the maximum allowed average speed by which the train realizing commercial stop $C_l \in C$ may go over node $v \in V_l$.

A solution consists of:

- a subgraph $G''=(V'',E'')$ with $V'' \subset V$ and $E'' \subseteq E'$ indicating the infrastructure to be installed. Let $G^e=(V^e,E^e)$ represent the complete augmented infrastructure, i.e., $V^e=V^0 \cup V''$ and $E^e=E^0 \cup E''$.
- for each commercial stop $C_l \in C$ a directed path $P_l \subseteq A_l$ starting at a node from $V(s_l^{start})$ and ending at a node from $V(s_l^{end})$. Let $V(P_l) \subseteq V_l$ be the set of all nodes on this path. Considering the signal position nodes R as separators, P_l can be partitioned into the ordered list of compound routes $L_l=(P_{l,1},\dots,P_{l,\lambda})$ with corresponding node sets $V(P_{l,1}),\dots,V(P_{l,\lambda_l})$. The length of route $P_{l,i}$,$i=1,\dots,\lambda_l$, is $l(P_{\ell,i}) = \sum_{v \in V(P_{\ell,i})} l_v$.
- for each (infrastructure) node $v \in V(P_l)$,$C_l \in C$, an (average) speed $speed_{l,v}$ that does not exceed the limit $maxspeed_{l,v}$. Consequently, the train takes time $T_{l,v}=l_v/speed_{l,v}$ for passing node v.
- for each route $P_{l,i}$,$i=1,\dots,\lambda_l$,$C_l \in C$, a reservation time slot $(T_{l,i}^{enter},T_{l,i}^{exit})$ in which the train will safely be able to pass this route.

To be feasible, a solution must satisfy:

- For each commercial stop $C_l \in C: \forall (u,v) \in P_l \rightarrow u,v \in V^e \wedge (u,v) \in E^e$, i.e., the infrastructure used in the chosen paths must exist or be installed.
- All constraints for realizing possible extensions (e.g., mutual exclusivity of some alternatives) must be adhered.
- The time slots of consecutive routes of a commercial stop overlap exactly by the corresponding safety margins.
- For each commercial stop $C_l \in C$, the earliest start and latest arrival times T_l^{start} and T_l^{end} are adhered, respectively.
- At each time, each node $v \in V^e \setminus R$ (i.e., except signal position nodes) may only be part of at most one reserved route.
- If the same train is used for two successive commercial stops, its arrival node at the station's track must be the same as the node where it leaves from later.

The objective is to find a feasible solution with minimum total costs $\sum_{v \in V''} c_v$.

The main difference between this formal model and the one presented in [4] is the introduction of different types of nodes in graph G. This allows for more flexibility and a more precise modeling, e.g., alternative nodes are used to distinguish between cases with or without building switches, depending whether the extension is applied (see Figure 1). In the previous model this was not possible.

3 Constructive heuristic solution approach

Our heuristic solution approach for TTBDRI consists of a construction framework in which an exact dynamic programming (DP) procedure is embedded for realizing the individual commercial stops. In the following subsection we present the DP, while Section 3.2 describes the construction framework.

3.1 Dynamic programming

The main idea for our DP is to use it for finding an optimal solution for just one given commercial stop $C_l \in C$. It will be applied iteratively until all commercial stops are realized and, possibly, a complete locally optimal solution is found. Therefore, from this point on, we will concentrate only on one given commercial stop $C_l \in C$ for which we want to find a cost-minimal realization.

To cover the aspect that a train may possibly start from and end at different platforms, we introduce artificial start and end nodes σ and τ, respectively, to the set V_l of the commercial stop C_l, i.e., $V_l' = V_l \cup \{\sigma, \tau\}$, and we set their costs and lengths to zero and maximum speed to one. Furthermore, we augment the arc set to $A_l' = A_l \cup \{(\sigma,s) \mid \forall s \in V(s_l^{start})\} \cup \{(s,\tau) \mid \forall s \in V(s_l^{end})\}$.

For every node $v \in V_l \backslash V_l^R$ we define a set of Y_v time intervals in which it may be possible to reserve node v for the train to pass it. Every such time interval of Y_v has length at least equal to the minimum reservation time needed for node v. This minimum reservation time is the sum of the time needed for travelling through node , the minimum time needed for travelling through any possible predecessor and the minimum time needed for travelling through any possible successor of node v.

For general principle of DP see e.g. [6]. Our DP stores labels $(c, T_R^{start}, T_R^{end}, t, \pi)$ for reached nodes, where

- π represents the preceding node;
- c represents the accumulated costs for the path from σ to v including c_v
- T_R^{start} represents the earliest time from which the reservation of the node v may start;
- T_R^{end} represents the latest time until which the reservation of the node may last and
- t represents the earliest arrival time at node v in time interval $[T_R^{start}, T_R^{end}]$

The initial label for node σ is $(0, T_l^{start}, T_l^{end}, T_l^{start}, \text{null})$.

The extend function $(c, T_R^{start}, T_R^{end}, t, \pi) \rightarrow (\bar{c}, \bar{T}_R^{start}, \bar{T}_R^{end}, \bar{t}, \bar{\pi})$ for considering as next step to go from node u to node v is: $\bar{c} = c + c_v$, $\bar{\pi} = u$ and for the calculation of $\bar{t}$, $\bar{T}_R^{start}$ and $\bar{T}_R^{end}$ and we need to distinguish the following cases:

- when $v \in V(s_l^{start})$, then for every $[T^{low}, T^{up}] \in Y_v$ the extend function returns a label with $[\bar{T}_R^{start}, \bar{T}_R^{end}] = [T^{low}, T^{up}]$ and $\bar{t} = T^{low}$;
- when $v \in V_l^R \cup \{\tau\}$, then the extend function returns the label with $[\bar{T}_R^{start}, \bar{T}_R^{end}] = [T_R^{start}, T_R^{end}]$ and $\bar{t} = t + l_u/maxspeed_u$;
- when $v \in V_l \backslash (V_l^R \cup V(s_l^{start}))$ we distinguish the following:
  1. when $u \in V_l^R$, then for every $[T^{low}, T^{up}] \in Y_v$ having a nonempty intersection with $[t, T_R^{end}]$ the extend function returns a label with $[\bar{T}_R^{start}, \bar{T}_R^{end}] = [T^{low}, T^{up}] \cap [t, T_R^{end}]$ and $\bar{t} = t$;
  2. when $u \in V_l \backslash V_l^R$, then for every $[T^{low}, T^{up}] \in Y_v$ having a nonempty intersection with $[T_R^{start},$

T_R^{end}] the extend function returns a label with $[\bar{T}_R^{start}, \bar{T}_R^{end}] = [T^{low}, T^{up}] \cap [T_R^{start}, T_R^{end}]$ and if $T_R^{start} \geq \bar{T}_R^{start}$, $\bar{t} = t + l_u/maxspeed_u$, else $\bar{t} = \bar{T}_R^{start} + [(t + l_u/maxspeed_u) - T_R^{start}]$

An extension is feasible iff the following two conditions hold. (a) The actual arrival time at node v has to be feasible, i.e., $\bar{t} \in [\bar{T}_R^{start}, \bar{T}_R^{end}]$. (b) A time exists at which the train can pass from previous route to the current one. This is expressed as $\bar{T}_R^{start} \in [T_W^{start}, T_W^{end}]$ where $[T_W^{start}, T_W^{end}] = [t, T_R^{end}]$ of the last signal position node on a path from σ to v.
Labels that are dominated by others can be removed. A label $l_1 = (c, T_R^{start}, T_R^{end}, t, \pi)$ dominates a label $l_2 = (\bar{c}, \bar{T}_R^{start}, \bar{T}_R^{end}, \bar{t}, \bar{\pi})$ iff the reservation time interval $[T_R^{start}, T_R^{end}]$ of label l_1 contains the reservation time interval $[\bar{T}_R^{start}, \bar{T}_R^{end}]$ of label l_2 and $c \leq \bar{c}$ as well as $t \leq \bar{t}$ with at least one of the latter two inequalities being strictly fulfilled.
Once when we have reached artificial end node τ actual solution is obtained by going backward until artificial start node σ is not reached. In every backward step we calculate the reservation time interval for visited node as well as appropriate speed used for travelling through it.

3.2 Construction heuristic

Our construction heuristic can be described by the following pseudo-code:

ConstructionHeuristic(S, C, i)
Given: partial solution S – a list of solutions for individual commercial stops; set C of not visited commercial stops; first not jet visited commercial stop i;
Output: complete solution S if there is such, incomplete solution otherwise;
for all $c \in C$ **do**
    if DP succeeded to find solution for the commercial c stop **then**
        S[i] found solution;
        if i < the total number of given commercial stops **then**
            ConstructionHeuristic(S, C\{c}, i+1);
        else
            return; //complete solution obtained
        endif
    end if
end for

In the first call of above function we set S to be an empty set and C to be the whole set of the given commercial stops.

4 Experimental results

All experiments were carried out on an Intel Core i7-860 processor on 2.80GHz with 8GB of RAM. The algorithm has been implemented in C++.
Test instances model existing infrastructure between Feldkirch in Austria and Buchs in Switzerland with all intermediate stations in Austria, Liechtenstein and Switzerland. F_B_scenario1 represents an infrastructure with a possible flying crossing extension at Nendeln station and two trains of type RailJet. F_B_scenario_2 consider possible extensions at Schaanwald, Nendeln and Tisis and use four trains, two S-Bahns and two RailJets. RailJet trains have only two stops, the start and the end station. S-Bahn trains, however, stop at every intermediate station between their start and end stations with the minimum dwell time of 30 seconds. Thus, for every RailJet we have one commercial stop, while for every S-Bahn we have 8 commercial stops in this particular case.

Table 1 Summary of the experimental results on a set of real-world instances

Instance	|V|	|E|	Number of		Objective value [mil. €]	Execution time [s]
			trains	commercial stops		
F_B_scenario1	171	176	2	2	24.890	0.079
F_B_scenario2	210	215	4	18	39.090	20.64

Table 2 Predefined arrival and departure times of used trains

Train type	Direction Feldkirch – Buchs		Direction Buchs – Feldkirch	
	Departure t.	Arrival t.	Departure t.	Arrival t.
RailJet	51'	06'	54'	09'
S-Bahn	48'	12'	48'	12'

5 Conclusions and future work

In this article we have presented a formal combinatorial optimization model for the integrated timetable-based design of railway infrastructure. We have then suggested a first heuristic approach for approximately solving this problem, which consists of a constructive framework in which an exact dynamic programming procedure is embedded for realizing individual commercial stops. Obtained results appear reasonable and encouraging but also indicate the need of further algorithmic improvements to solve more complex scenarios more effectively. In future work we aim at applying more sophisticated hybrid metaheuristics to obtain better solutions with prolonged computations (see e.g. [5]), but also exact techniques based on mathematical programming methods like column generation and Benders' decomposition.

References

[1] Caimi, G., Laumans, M., Schüpbach, K., Wörner, S. & Fuchsberger, M.: The periodic service intention as a conceptual framework for generating timetables with partial periodicity. Transportation Planning and Technology, 34(4), pp. 323–339, 2011.

[2] Lichtenegger, M.: Der Integrierte Taktfahrplan: Abbildung und Konstruktion mit Hilfe der Graphentheorie, Minimierung der Realisierungskosten. Eisenbahntechnische Rundschau, 40, pp. 171–175, 1991.

[3] Schöbel, A. & Besau, G.: Timetable Based Design of Railway Infrastructure. In EURO-ZEL 20th International Symposium, June 2012.

[4] Schöbel, A., Raidl, G. R., Grujicic, I., Besau, G. & Schuster, G.: An optimization model for integrated timetable based design of railway infrastructure. In Proceedings of the 5th International Seminar on Railway Operations Modelling and Analysis – RailCopenhagen 2013, pp. 765-774, IAROR, 2013.

[5] Raidl, G.R., Puchinger, J., Blum, C.: Metaheuristic hybrids. In M. Gendreau and J. Y. Potvin, editors, Handbook of Metaheuristics, volume 146 of International Series in Operations Research & Management Science, pp. 469–496. Springer, 2nd edition, 2010.

[6] Bellman, R.: Dynamic programming, Princeton University Press, Princeton, New Jersey, 1957.

REGIONAL RAILWAYS: TIMETABLE-BASED LONG-TERM INFRASTRUCTURE DEVELOPMENT

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Abstract

In regional traffic, we are confronted with an ageing railway infrastructure with both ridership decrease and a poor structural condition, interlocked mutually: Investments are hard to justify because of low traffic loads, but service improvements are bound to fail due to the inadequate infrastructure. Therefore, a smart, iterative design process must be started. Taking into account future structural changes and demand shifts, a target timetable is created. First, a demand model for the planning area is set up and calibrated against the current traffic flows of both public transport (train and bus) and individual transport. Second, a prediction of future population changes and structure developments (settlement and road infrastructure) is set up and woven into the demand model. Third, the demand model is fed with the future data and the current timetable to check the development without measures. Fourth, a sensitivity analysis of competing timetable models is checked against the demand model; a detailed analysis of these results is then taken to optimise the best-ranked timetable model. Next, the design of a more detailed timetable can start. By means of an integrated timetable model, the operationally necessary riding times between hubs as well as along the track are defined as well as the rough location of two-track sections. These parameters are then taken over to check infrastructure as well as vehicle options to achieve these goals. In a project-wise analysis, the optimal combination of infrastructure measures is then obtained to be able to run the predicted timetable. Since this timetable has been put beyond dispute beforehand, every single infrastructure measure can be justified including all consequences that result from its implementation. This results in a step-by-step upgrade plan and a definitely optimal, long-term and timetable-based infrastructure development perspective.

Keywords: regional traffic, railway, timetable, infrastructure, target network

1 Introduction

Regional Railways generally face two problems: decreasing ridership and an ageing, mostly inadequate infrastructure. Additionally, these two problems are highly interlocked, since public spending on railway lines on the decline cannot be justified. This results in unattractive conditions for passengers, further decreasing the ridership and leading to the ever-so-often cited vicious cycle. Most examples of successful regional railway lines around the world arose either in times of general readiness to invest or as a result of lobbying of strenuous pressure groups. However, neither of these prerequisites is guaranteed and neither reflects the actual potential of regional railway lines, both leaving the commitment for regional railway traffic and its long-term development highly subjective. What both communities and railway infrastructure operators need is a clear business case and a long-term development perspective to allow for a transparent decision concerning future service offer as well as infrastructure upgrades.

2 Objectives

As mentioned above, regional railway traffic is often not only confronted with a limited commitment to investment, but also with an unstable forecast concerning its long-term perspective. Due to the long service lives of railway infrastructure, this combination usually leads to a mere persistence at the status quo, leaving necessary re-investments unaccomplished, hampering upgrade measures and rendering significant service offer modifications impossible.
The process presented here covers this special situation of regional railways. The objective is a long-term infrastructure perspective based on a target timetable and an economically optimal set of measures to achieve that goal. The timetable is decided upon at an early stage and put beyond discussion by means of demand modeling. Since the target is clear and all measures align with it, every single investment and its timeframe can be justified with the long-term development plan – and also, political influence can be channeled along transparent decision parameters.

3 Methodology

The methodology presented here is an adaption of a classical iterative engineering design process [3], suitable for regional railway lines. It requires a demand model, a population and structure prediction on community level, a detailed knowledge of current railway infrastructure conditions and a thorough base for railway upgrade measure design at several fields of engineering. The main features of this process are the interdisciplinary feedback loops, comprising demand modeling, timetable construction and upgrade measure design in the fields of infrastructure projects, operational measures and vehicle characteristics.

3.1 Demand modelling

The approach requires a well-kept and calibrated multimodal demand model capable of modelling modal shift and sensitive to timetable changes. This model is fed with the current population, structure and timetable of all means of public transport. It is calibrated upon both ridership of public transport and road traffic load profiles. Furthermore, it is checked against actual travel time and travel distance distributions. The latter is of great importance especially in regional transport, as it accounts far more for the load profile compared to urban transport.
It should be noted here that the creation and calibration of the demand model as well as the quality of the input data is crucial for the whole design process. This is why at this stage, trade-offs concerning workload or project speed pose a great danger for the quality of the whole process.
An important aspect in the demand model is the need for a service responsive demand recalculation, meaning a recalculation of the complete demand matrix rather than a mere redistribution of static OD matrices for each mode.
The demand model plays an important role throughout the whole process and will be fed with new information in several project stages in order to produce new input data for the other fields of design.

3.2 Population, structure, and infrastructure prediction

Since the demand model is calibrated to model the status quo, but will be used to model the future demand and ridership, a set of predictions is included in the demand model. A simple traffic volume increase, as often used, does not reflect the problem at all: Especially in rural areas, traffic volume will depend upon a wide range of demographic characteristics, such as age distribution, car ownership and education, which will in many cases result in a traffic volume decrease rather than an increase, as well as in a shift of peak hours.

Figure 1 Workflow of the methodology

Therefore, population predictions must include at least an age distribution; structure predictions must comprise workplaces, schools, higher education, and public offices, thus reflecting future administrative changes. Infrastructure predictions need to concern all areas of traffic not touched by the upgrade plan (road network extensions/modifications, upgrades/closures of surrounding lines, etc.) to ensure that all comparisons refer to the planning horizon.

3.3 Case design

Since all upgrades target at a future point in time, the cases taken into account for comparison also need to reflect this future – representing what would happen if no action was taken. The design cases are based thereupon and represent changes in the service offer to be compared with the base case. All comparisons are made within the demand model, so the complete demand matrix and the modal split need to be recalculated at every step. This is, in planning areas typical for regional railway lines, a time-consuming process. There is, however, no alternative to this step, since a complete demand recalculation is not only desirable, but crucial for the iterative design process.

3.3.1 Base case

The base case, upon which all other evaluations are to be based, needs to reflect both the future population and structure, but also the future infrastructure apart from the regional railway line in question. This leads to a case with all circumstances of the future, but with the current, meaning unchanged, regional railway infrastructure and timetable. If the current timetable and infrastructure do meet the future demand, this means a no-action strategy is sensible; if not, the reaction of passengers to structural changes will be evident.

3.3.2 Model timetables

An examination of passenger behaviour then leads the way for model timetables. The purpose of model timetables is not to deliver an operationally feasible timetable, but to derive trends from extremal approaches to tackle the identified demand problems. A tight station spacing, for example, suited for a regional area coverage, contradicts the need for a competitive riding time. Also, the ridership increases with a denser interval, but there is a certain point after which the additional costs for an improved offer outweigh the ridership increase.
Model timetables should differ in no more than one property each – interval, station spacing, riding time, product structure – to allow for an isolated comparison of trends. This, in turn, leads to an exponentially growing number of comparisons, since all trends are derived from pairwise comparison. Therefore, the differences between the model timetables should be as extremal as possible. The result of the model timetable comparisons then leads to two further steps: the selection of the key parameters of the model timetables that proved successful in the demand model leads the way to the construction of an operationally feasible target timetable. Meanwhile, the results obtained in the demand model from comparing the model timetables are used for a sensitivity analysis. This is to retrieve critical values to be obeyed by the target timetable.

3.3.3 Feasible timetable solution

The purpose of the model timetables was the demonstration of demand and ridership effects of various timetable approaches. An operational feasibility would not only be unnecessary, but also potentially handicapping: since the scope of upgrade measures to obtain operational feasibility is highly subjective, such a timetable could also obscure possible improvements. The next step, creating an operationally feasible timetable, includes as many characteristics as possible from the successful model timetables of the preceding step, while the previously identified critical values serve as limit values. These limit values guarantee that the main goals (ridership, modal split) will also be reached by the operationally feasible timetable. The feasible timetable is constructed within a standard timetabling routine. However, restric-

tions such as switch speeds, level crossings or slow orders need to be removed in the design process. The maximum speeds on the open track take into account the alignment only, and the (virtual) existence of double-track sections is handled flexibly according to the needs of the timetable construction.
This timetable solution aims at constructing a framework for a target timetable when it comes to riding times between hubs, necessary transfer connections and the rough location of double-track sections. While it would, in principle, be possible to fix the exact location of all double-track sections as well as alignment and other measures, it is not the goal at this very stage. Since the exact measures that are to be contained in the upgrade plan are highly interdependent, any fixed measure at this stage will lead to undesired restrictions.

3.4 Feedback loop

As noted already, the feedback loops between the demand model, the timetable construction and the measure design play a vital role in this design process. After the cases have been designed and the model timetables have been evaluated in the demand model, an important feedback loop is started: taking all results obtained, the key success factors of the various timetable approaches need to be worked out. After narrowing down the extremal values of the model timetables, the aforementioned critical values can be obtained, such as the maximum acceptable riding time and the optimal interval as well as the (both positive and negative) impact of station closures or relocations. This is done by modifying the parameters in question until the target performance indicators (modal split, modal shift, ridership, etc.) are reached. This information is necessary for supplying the feasible timetable solution with limit values not to be exceeded.
The second feedback loop takes place after the feasible timetable solution has been completed. In the demand model, the performance of the feasible timetable is evaluated and checked against the target performance indicators (modal split, ridership, passenger load, etc.) as well as the performance obtained with the model timetables. Clearly, the feasible timetable will feature certain compromises, since not all target aspects will be met equally by the feasible timetable. Nevertheless, the key features obtained from the sensitivity analysis must not be exceeded so as to remain attractive enough for passengers to meet the target performance.
If the demand model shows a performance too poor to be accepted, the feedback loop ensures that modifications can be made in the timetable without lost work in the measure design.

3.5 Target network features

Clearly, the feasible timetable solution will be close to the final target timetable. It will, however, still offer enough flexibility to allow an optimisation of the infrastructure measures. What is needed for this next step is a definition of the target network features to allow the target timetable to be operable.
The target network features to be retrieved are the target riding time between hubs as well as between stations/double track sections, the rough location of double track sections and the required station layout (with an emphasis on parallel approaches/exits and cross-platform transfers). Since there is a high grade of interdependency between almost all possible measures, the exact location of double-track sections and the exact station layout can only be determined within the measures design process.

3.6 Infrastructure, operational and vehicle measures

Finally, all target network features need to be achieved via an optimal arrangement of measures. In the judgement and selection process, the methodology of Uttenthaler [1] was chosen. This simple approach allows a flexible assessment of measures. All measures in a section in question, yet grouped when necessarily combined, are listed with their individual riding time benefit. Since the necessary riding time reduction is known, the addition of potentially every measure combination is possible to achieve the target riding time. The most cost-efficient and/or most beneficial set of measures allowing the target riding time is then selected and included in the target network measures. Certain measures, such as the electrification of a line or vehicle characteristics, affect the whole line rather than individual stretches, so they have to be added or subtracted along the whole line. Alongside traditional alignment measures, there is a wide variety of measures that allow riding time reductions at lower costs with comparable benefit [1]:

- **Travel time reserves:** In an integrated timetable, travel time reserves are usually bigger than in non-cyclic timetables, since disruptions lead to either lost transfers or a network-wide spread of delays. Therefore, riding time improvements at the cost of riding time reserves should be avoided completely or be only of temporary nature.
- **Vehicle dynamics:** One often untouched parameter of a railway network is the vehicles' capability of better acceleration or deceleration. This is especially important in regional railways, since with short station spacing it makes up a great percentage of the riding time. An electrification of railway lines, which leads to a significant improvement of the vehicle dynamics, can be justified via the necessary riding time reduction. In stations, however, a better acceleration leads to a higher speed at the station gridiron and will therefore require a change of the switch geometry. This, in turn, usually can be obtained at manageable costs, especially in the long run.
- **Number of stops:** Regional railways normally feature a dense sequence of stations for a area coverage. However, railway stations often have been built well outside settlements upon construction of the railway line and have not been repositioned since. Additionally, the current and future mobility structure with a high grade of intermodality decreases the need for small stops with a limited amount of passengers. Compared to the amount of time potentially gained by alignment measures on the comparatively short stretches of open track in regional railways, a stop left out will lead to a fair amount of riding time decrease.
- **Stopping time:** regional railways typically do not feature many stations with great passenger volume. Therefore, the practical time needed for stops is often less than the typical design value for stopping time in regional railways, 30 seconds. Shortening that time to the actual needs results in further riding time decreases, albeit in a small scale.
- **Uncompensated sideways acceleration:** Regional railways typically run with low axle loads due to light vehicles. The limit values for free sideways acceleration, however, typically cover a range up to heavy freight trains. Without tilting technology, uncompensated sideways accelerations of up to 1,1 m/s^2 are acceptable internationally, with the light vehicles not affecting the superstructure more than considerably heavier trains at lower sideways accelerations. This leads to a better exploitation of the alignment and riding time decreases in similar dimensions as alignment measures.
- **Switch geometry:** train stations are often equipped with tracks long enough to serve freight trains. Together with the standard geometry of switches, this often allows no more than 40 km/h for a long stretch before the platform. If the switch is optimised to the approach speed to be expected at the very location of the switch, the full track design speed can be exploited over a maximum stretch.
- **Level crossings:** regional railways usually feature a great number of level crossings without technical safeguarding. This affects the possible track speeds for security reasons. Therefore, a technical upgrade of level crossings has a significant effect on the riding time on regional railways.
- **Station layout:** While all other measures aim at a direct decrease of riding time, the station layout cuts the required riding time reduction. The main application of a redesigned station layout are transfer stations, since a parallel departure/arrival shortens the headway between two trains and therefore the required riding time reduction of one of the trains. At stations with train crossings, pedestrian level crossings prohibit parallel approaches, so a technical safeguarding or an underpass reduce the required riding time reduction.

- **Signalling:** alike the station layout, signalling affects the possible headway and therefore the required riding time reduction. Signalling does, in practice, often affect the main speed on the open track, too, but since any changes in track speed would be futile without an adaption of the signalling system, we assume that this is carried out anyways.
- **Alignment measures:** Finally, alignment measures form the traditional, but usually most expensive riding time reduction measures. Apart from classical alignment projects, the application of optimised curve design for small angles and small radii leads to a significant increase of track speed without much deviation from the old alignment [2].

3.7 Step-by-step upgrade plan

With the final target network defined and the set of measures fixed, the next step is a bundling of measures into sensible packages. Therefore, a set of intermediate timetables is defined, so that the logical succession of the upgrade measures can be derived from these intermediate steps. In accordance with the upgrade plans of surrounding railway companies and the succession of these projects, the amount of measures taken too early can be reduced to a minimum and key projects can be prioritised.

4 Application

The methodology presented here is currently applied to the network of Graz-Köflach Railway (GKB) in the south west of Graz. The design horizon is the year 2025, which aligns with the target network of the Austrian Federal Railways (ÖBB) upon completion of the Semmering Base Tunnel and the Koralm Railway Link, both heavily affecting the planning area. The following data was obtained from several sources and is used as a design basis:
- The company strategy of GKB;
- The national, regional and local transport strategies of the republic of Austria, the province of Styria and three districts touched by the railway network;
- The national target timetable of the Austrian Federal Railways with the integrated timetable hub at Graz;
- The population and structure predictions of the province of Styria;
- The planned road network extensions of the province of Styria.

The demand modelling process and the timetable design have already been finished, while the measures planning process is still underway. So far, it has been shown that:
1. the population and structure predictions show a growth and a concentration of the population around the cities, while the more remote areas face a further population decrease;
2. the various timetable models developed show a clear preference to close down several smaller stations to allow for a competitive riding time between the bigger cities;
3. the demand model allows for a level of timetable judgement as detailed as necessary for regional railway networks, especially when it comes to a sensitivity concerning interval and stopping policy;
4. the necessary measures can be reduced to a few double-track sections, the technical safeguarding of level crossings and a big emphasis on switch geometry in stations;
5. the step-by-step upgrade plan is an effective and absolutely objective tool for negotiations about the future of the regional railway network.

Detailed analyses of the measures necessary for the target timetable are currently being conducted. The project is due to be completed by summer 2014. When finished, the railway network is supplied with a long-term upgrade concept without the need for short-handed negotiations about singular measures.

5 Conclusions

The approach presented here is, in general, nothing particularly new, since any design process in engineering will be based upon a design load and the dimensioning thereupon. However, the timetable oriented design process is still rather new in railway infrastructure, the use of demand modelling in an iterative railway infrastructure design process has not been carried out in this degree of profundity before and especially not in regional transport.
There is little chance that the upgrade plan fails at a later stage due to a demur concerning an earlier one, since (i) the demand has been modeled on a solid basis; (ii) the target timetable has been put beyond disussion beforehand; (iii) the measure set is the result of a thorough assessment process; (iv) the succession of the measures follows the outside circumstances and a logical combination of measures; and (v) all feedback loops occur within the design process already. This results in a clear business case as a long-term decision basis for the future of regional railway lines.

References

[1] Uttenthaler, H.: Grundlagen eines auf einem integrierten Taktfahrplan basierenden Infrastrukturausbaues am Beispiel Zentraleuropa. Diploma Thesis, TU Graz, 2010

[2] Scharler, H.: Weinzinger, G.: Cost Savings in Track. Added Value by Advanced Track Alignment. Romanian Railway Summit, Bucharest 2008

[3] Sonntag, H.: Linienplanung im öffentlichen Personennahverkehr. Dissertation, TU Berlin, 1977

INTEGRATED PERIODIC TIMETABLE BASED CONCEPTS IN HUNGARIAN NATIONAL TRANSPORT STRATEGY

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Abstract

This paper is about the methodology of defining projects for the National Railway Development Concept, which is an essential part of the Hungarian National Transport Strategy referring to the time perspective of EU White Book planning for medium (2020), long (2030) and strategic framework term (2050). The concept and the methodology are based on service concepts and integrated periodic timetables. The goal of the strategy is to improve performances of the transport network, to achieve a higher modal split ratio for public transport, by increasing the competitiveness of railways and of the entire public transport system.
The first part of the paper is about the passenger demands. It analyses competitiveness of the current railway network, and it points out the bottlenecks of the network – where the capacity is not enough, and the parts, where the competitiveness is poor. It defines two kinds of necessary interventions: increasing competitiveness and eliminating bottlenecks. The second part is about the current integrated periodic timetable and its development concepts which are able to increase the competitiveness in short, medium and long term. The timetable development concepts can show, where and what the real bottlenecks of the network are. By these concepts we can define the exact infrastructure development projects which are necessary to achieve the main goal of our strategy. European and Hungarian taxpayers could also benefit from using this methodology, by making it possible to avoid inefficient investments. The last part of the paper gives a detailed overview of the proposed projects embedded in the strategic aims. The proposals could ensure an efficient use of the EU sources in the budget period 2014–2020, by increasing the competitiveness of railways.

Keywords: integrated periodic timetables, transport strategy, public transport system

1 Introduction

First we have to know what our aim is. The aim has to be based on passenger demands. For knowing what the passengers want, we analysed the competitiveness of the current railway network, which indicates the bottlenecks of the network – where the capacity is not enough, and the parts, where the competitiveness is poor. It defines two kinds of necessary interventions: increasing competitiveness and eliminating bottlenecks.

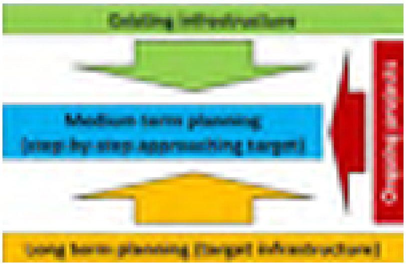

Figure 1 Flow chart of the process

2 Analysing the market and competitiveness

2.1 The size of the market

Before getting to the network, we analyse how many railway passengers we have in each relations, and most importantly, the total number of passengers travelling between our destinations, including passengers using other modes of transport. From this data, we can estimate how many new railway passengers could be the maximum target of the development. For estimating it we are using data from the Hungarian Central Statistical Office about numbers of journeys in total between major Hungarian cities and Budapest.

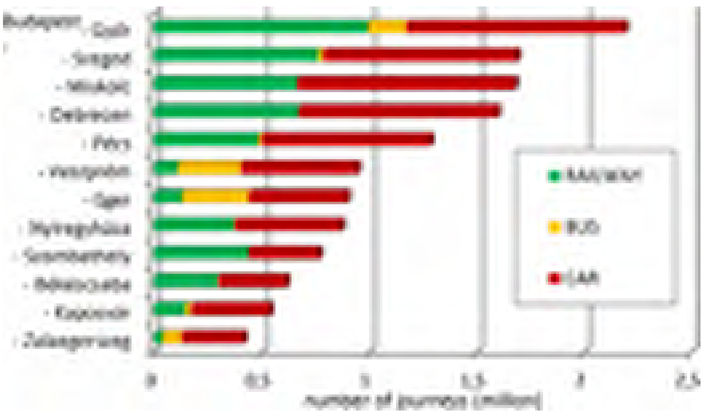

Chart 1 Journeys between Budapest and the major cities[1]

2.2 The number of railway passengers

The diagram above shows the total number of the passengers. To have a more detailed view, we used the data from MÁV-START (passenger railway company) ticketing statistics, to see how many railway passenger we have. To estimate the railway's competitveness between the major cities and Budapest, we've looked at how often an average citizen travels between their cities and Budapest (total yearly number of journeys divided by the city's population). The current railway services are more successful where from there are more railway passengers to Budapest. The differences between the major cities are huge.

Figure 2 Railway journeys to Budapest per inhabitants [1] [2]

2.3 Parameters of the railway services

To see the reason for these huge differences, we checked the parameters of each connection. We've compared the journey time and the frequency of the service to the travel time by car. As a result, we got a variable which combines the frequencies and the journey time, and named it 'average travel time'; this is the variable which is comparable with the journey time by car. We calculated the average travel time by adding the journey time and the average waiting time which is the half of the frequency.

2.4 Correlation between competitiveness and numbers of passangers

It is suspected that there is a correlation between the competitiveness and the success of the railway services, therefore, we checked how strong it is. Comparing the railway's average travel time to Budapest with the percentage of travel time by car, and the railway journeys to Budapest per one inhabitant in a year, the correlation coefficient is -0.84 which shows a very strong correlation between the two variables. The diagram below shows the correlation in two dimensions, it's visible that all the points are pretty close to the line.
There is one more correlation visible on the diagram: the cities closer to Budapest have more journeys per inhabitants. This fact makes even stronger correlation between the competitiveness of the journey times and the success of the railway services. The only exception is Szeged, but the railway services between Szeged and Budapest are traditionally very good. This fact shows that travel time and frequency are very important, and there is a strong correlation between this, and the number of passengers, but this is not the only factor.
From that point it's clear that the target of infrastructure developments should be to decrease travel time, and to increase frequency. The chart also shows the directions, where competitiveness is poor.

Figure 3 Correlation between travel times and number of passengers [1] [3]

3 Target infrastructure

Now, we know our target from the passenger's point of view. Our next task is to elaborate our target railway timetable structure, as a backbone of an attractive transport system. Our aim is to create such a railway network, where public transport is competitive with driving, and not only with a capital city as destination. ITF was designed to reach desired travel times in a cost-effective manner. If applied correctly (at least hourly), on a network with good connections, integrated periodic timetables require minimal infrastructural developments [4].

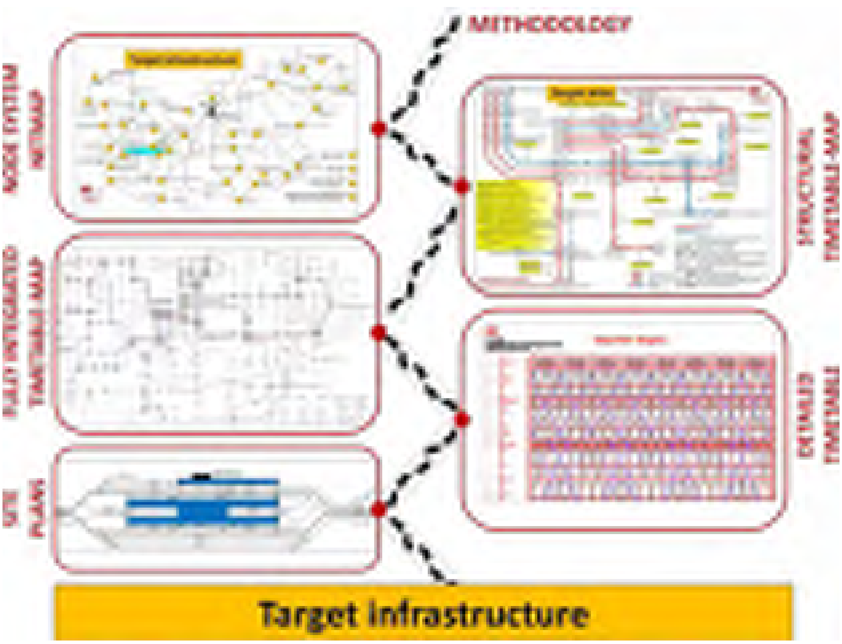

Figure 4 Methodology of developing an efficient transport system

3.1 Methodology

Fig. 4 shows the steps necessary to find out the requirements for such an infrastructure. Firstly, we need to define the types of timetable hubs (60, 30, 15 minutes), according to the physical position of the transport nodes (bigger cities) and to the target travel times between them. In this stage, structural problems of current infrastructure may appear, highlighting the need to replace current junctions to different, more suitable places.
Secondly, we need to define long-distance traffic in details to the previously designed system. The output is the long-distance periodic map (Fig. 5), which shows the necessary infrastructural developments needed to reach the target travel times.

Figure 5 Structural timetable-map for target infrastructure

Based on this long-distance periodic map, we can construct a detailed map, showing our transport network as a whole, including the relevant bus lines that support our system. With the help of this fully integrated periodic map, the detailed timetable can be constructed, showing the required improvements in details (track speed increase, railroad switches, and railway platform designs for transfers).
If we go through the steps above, we get such an infrastructure (needed infrastructure connections and track speeds can be seen on the right side of Fig. 6.), which enable us to design a competitive public transport system (existing and target running times of fastest trains could be seen on left side of Fig. 6.), but does not contain such expensive elements, where capacity utilization is not effective.

Figure 6 An efficient target infrastructure for the Hungarian transport system in our suggestion

4 Reasonable infrastructure development projects for the Hungarian railway system in the 2014-2020 financial period

Knowing the target infrastructure helps us to create a list of current development projects that are directed to one specific goal, and their individual results amplify each other instead of weakening the return indicator. The order of implementation may vary among the projects, but certain priorities need to be set, so that the ones with most benefits would be completed first. As part of our work, we've compiled a development package, based on the sources available in the financial period of 2014-20. This package does not only comply with the requirements of EU subsidies, but also contains coherent elements, that help reaching quality improvements for the most possible passengers.

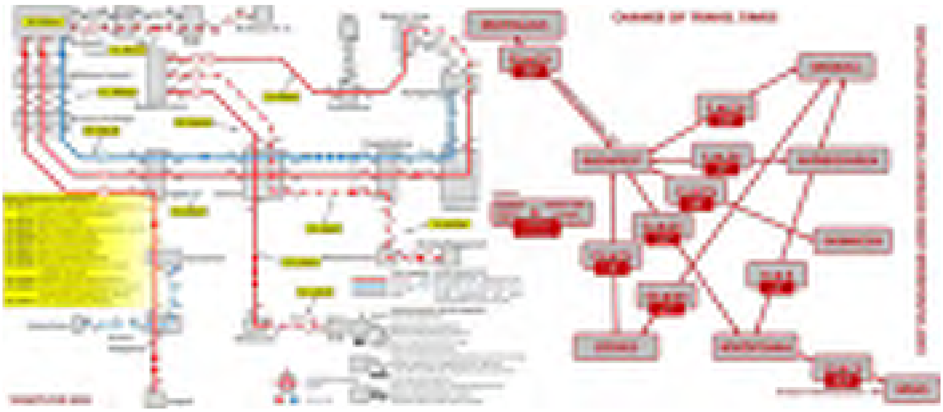

Figure 7 Timetable structure in Eastern Hungary for 2020

4.1 Eastern Hungary – Eliminating bottlenecks

In Eastern Hungary, railway traffic is already on the border of competitiveness, with an infrastructure that is rather worn out. Due to its competitiveness, most of our revenue from passenger rail is generated in this region; therefore, preserving the schedule structure from possible negative effects of infrastructure deterioration is top priority. Instead of recent, detailed, but very fragmented modernisation projects, we've proposed an implementation of a more comprehensive development package, containing only the most essential works needed on Eastern Hungary's main lines. MÁV is already working on the detailed plans.

4.2 Transdanubia – Increasing competitiveness

In Transdanubia (the western part of the country) railway services are much less competitive, with significantly less passengers in general (figure 2). The main target in this region is to increase competitiveness. To reach this goal, developments are necessary in the following directions, starting from Budapest: to Veszprém, to Zalaegerszeg, to Pécs, to Kaposvár and to Szombathely. Infrastructure developments are recommended on lines between Budapest and Pécs, Székesfehérvár and Veszprém, implementing ETCS on lines between Boba and Zalaegerszeg, and between Budapest and Székesfehérvár. These projects should be focusing only the parts which are necessary to reach our goals; this is the most efficient way of using our resources.

Figure 8 Timetable structure in Transdanubia for 2020

5 Conclusion

Among few other projects, all our major recommended projects got accepted, and they are all in the National Transport Strategy. That means our sources for infrastructure developments will be used in an efficient way in the following EU budget periods.

References

[1] Source of the number of journeys: Hungarian Central Statistical Office data service for National Transport Strategy

[2] Source of the number of inhabitatnts: Hungarian Central Statistical Office Population census 2011 – Preliminary data, http://www.ksh.hu/apps/shop.kiadvany?p_kiadvany_id=10205&p_temakor_kod=KSH&p_lang=EN, 05.09.2013.

[3] Source of the railway travel times: Hungarian Official Timetable of the 2010/2011 timetable period.

[4] Borza, V., Földiák, J.: Integrated Periodic Timetable in Hungary – experiences, help for vision, 2nd International Conference on Road and Rail Infrastructure – CETRA 2012, 5–7 May 2012, Dubrovnik, pp. 831-840.

A NEW APPROACH FOR DEFINING THE IMPROVEMENT PLANS OF RAIL NETWORKS

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Abstract

Railway operations are the result of the complex interaction among infrastructure and signaling, rolling stock and timetable. This interaction is further complicated by a number of human factors and other unpredictable phenomena and stochastic disturbances. The key role in this complex mix is played by the timetable, which allows smooth operation by shaping the services to fit the demand and the characteristics of the network and the rolling stock.
Different strategies might be implemented: in some countries the new lines and stations are very flexible towards different timetable concepts, while in others, such as in Switzerland, the are lines specifically designed for a given timetable. Both extreme approaches show significant drawbacks: a higher flexibility is normally obtained at a significant cost, while a very rigid infrastructure might require remarkable investments to be adapted to different needs.
In this trade-off a certain balance might be obtained by analyzing several timetable and infrastructure configurations. However, this task appears particularly time-consuming and strictly related to the experience of timetable planners: thus, normally very few scenarios are considered.
To fill this gap, by significantly reducing the time required to prepare a scenario, a new approach was developed. It is based on an automatic timetable generation tool that allows quickly creating timetable drafts on a quite detailed infrastructure model. The model and its application to the Norwegian rail network will be presented in the paper.

Keywords: railway capacity, timetable reliability, stochastic simulation

1 Introduction

Railways are struggling to compete with the other transport modes, which have the advantage of a remarkably higher flexibility in the operations. To increase the competitiveness of railways during this economic downturn it appears important to be able to select the investments that are effectively necessary to improve the quality of service where the demand might effectively increase as a consequence of the improvements.
Railway operations are the result of the complex interaction among infrastructure and signaling, rolling stock and timetable. This interaction is further complicated by a number of human factors and other unpredictable phenomena and stochastic disturbances. The key role in this complex mix is played by the timetable, which allows smooth operation by shaping the services to fit the demand and the characteristics of the network and the rolling stock [1].
Different strategies might be implemented: in some countries the new lines and stations are very flexible towards different timetable concepts, while in others, such as in Switzerland, the are lines specifically designed for a given timetable. Both extreme approaches show significant drawbacks: a higher flexibility is normally obtained at a significant cost, while a very rigid infrastructure might require remarkable investments to be adapted to different needs.

In this trade-off a certain balance might be obtained by analyzing several timetable and infrastructure configurations. However, this task appears particularly time-consuming and strictly related to the experience of timetable planners: thus, normally very few scenarios are considered. In this paper an approach is presented, in which timetables are generated semi-automatically using a timetable generation algorithm. The obtained timetable can be refined by the planner, and its core parts can be tested using microscopic simulation to estimate robustness under real conditions. Different scenarios are easily created preparing the infrastructure model and then running the timetable generation algorithm again. In this way it become possible to create and compare several scenarios even under the normally constrained time.

2 Approach

In the proposed approach, the rail planning process is viewed as a loop, whose central element is a timetabling algorithm, which allows automatically creating a timetable that contains the expected services, scheduled considering the constraints due to the network (including its interlocking and signalling system).

Figure 1 Block diagram of the approach

The planning process (Figure 1) starts with the definition of the required services and of the preliminary infrastructure. The services are defined by their origin and destination, stopping pattern and type of trainset. Some additional parameters are required by the timetabling algorithm as additional costs to be used in the objective function: a cost for increasing the running time, adding a stop, reducing the buffer time between two services and breaking a connection. The infrastructure is represented using a mesoscopic graph, which offers an ideal compromise between macroscopic and microscopic graphs, and appears very suitable for timetable generation. On the mesoscopic graph, a timetable is created as the best solution obtained using an automatic timetable generation algorithm, specifically designed to allow planners to interact with its results. The first result of the timetable generation is a feasibility check. In fact, the algorithm might be unable to schedule all required services.
In this case, the planner might try to remove some constraints to the timetable (connections, maximum running time, etc.) and repeat the generation or – should this not prove possible or sufficient – improve the infrastructure.
Supported by the infrastructure saturation levels shown graphically by the blocking times steps, and considering the list of services that could not be scheduled, the planner can identify a (set of) network improvement measures. They are easily implemented in the mesocopic model, and a new timetable can be generated on it. This “inner loop” is repeated until a timetable proves feasible.
The robustness of the feasible timetable can now be verified, to assess whether the services could be operated with satisfying reliability levels, especially on the most complex or densely-used sections of the network Microscopic simulation is used to perform, obtaining as output the same delay and punctuality indicators used by rail operators to measure real delays.

Should the timetable not prove robust, planners would be required to further improve the infrastructure (or – when possible – remove some timetable constraints) and the perform the simulation again. The block diagram of the approach is presented in Figure 1. The key elements of the approach are presented in detail in the following sections.

3 Infrastructure model

The infrastructure model used as a basis for planning the timetables is a key element in the planning process [1], since it must be defined quite easily but also represent all characteristics of the network that are relevant for calculating the running times and the occupation time of each train. Compared to a macroscopic model, in which stations are represented in a simplified way and fixed running times are used, a microscopic model appears significantly more accurate in the estimation of the key parameters used in timetable planning, such as the running and blocking times.

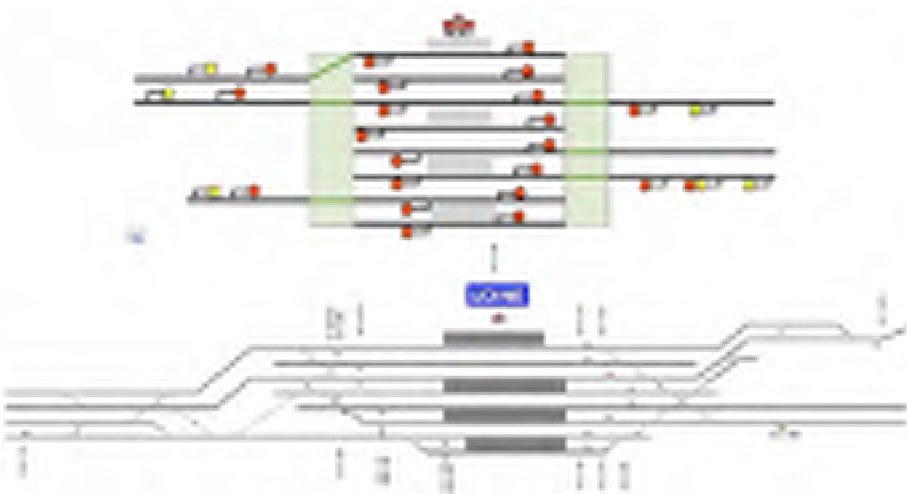

Figure 2 Mesoscopic (above) and microscopic model of a station

Most algorithms that solve the TTP problem are based on a macroscopic model, while some others, such as [4] and timetabling tool DONS [5], introduce a two-level approach. They use a macroscopic model to create draft timetable that will subsequently be checked for feasibility on a microscopic level for the principal station areas of the country. In any case, none of the two works consider the blocking times.

In order to combine the advantages of micro and macroscopic models some authors proposed algorithms for an automatic generation of a macroscopic model based on the corresponding microscopic one [6]. In the same framework, a microscopic model less detailed than the conventional ones was used Caimi [7] to represent the station areas.

In this work a mesoscopic model is introduced. It includes most of the accuracy of a microscopic model, but it also maintains a reasonable complexity. Its aim is to allow the generation a network-wide timetable in a reasonable time without the necessity of resorting to two-level approaches -level approach. Similarly to the macroscopic models, the network is rigidly separated into stations and line sections.

The station features of our mesoscopic model are the station tracks, the line tracks of the lines converging to the station, the distant and home signals at their distance from the station building; and a switch region at each side of the tracks (except, of course, the dead-end stations): each line is connected to all tracks; a set of matrices contains the possible and

impossible routes and their compatibility. Figure 1 shows the mesoscopic model of a station. The line features of the mesoscopic model include all signals, the speed limits, the gradients and the curve radii.
The running time of each train is calculated by solving the motion eqation, considering the exact train routing and signaling system as well as the characteristics of the rolling stock. Together with the running time on the default track, also the running times within each station of all possible routings are calculated. The result is model in which the running times are estimated with the same accuracy of the microscopic ones, while the blocking times and the definition of conflicts appears slightly simplified.

4 Timetable generation algorithm

Key factors for the acceptance of a software tool such the Timetable Planning Software (TTPSW [2]) are its perceived usefulness and the perceived ease of use. Within this framework, the final user, i.e., the timetable planner should not feel losing the control of the planning operations in a domain where he assumes to have some critical informal knowledge that cannot be easily transferred to an information system. In addition, as in a real environment the criteria defining the optimality of a timetable are often quite fuzzy, the final user should also be able to influence the structure of the timetable that the TTPSW picks up among the feasible ones.

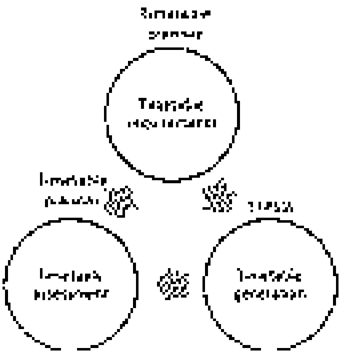

Figure 3 Timetable generation circle.

The aim of TTPSW is to allow the planner to iteratively generate different timetables in a reasonable time in order to assess them also in the light of the of its informal knowledge and to provide new (or different) constraints and objectives to the TTPSW for the generation of a new round of timetables. The algorithm tries to define a (sub)optimal timetable which includes each of the families of trains given as inputs. The timetable is generated by implementing a local search heuristic whose pseudocode.
The heuristic iterates within two cycles: an internal one and an external one. At each iteration a new timetable is generated and its cost is assessed. If this cost is less than the cost of the currently best timetable, the currently best solution is updated with the newly determined timetable.
The algorithm generates a new timetable at each iteration according to a greedy procedure, which defines the schedule and the associated penalty costs for the trains of a family at a time. The families are scheduled according to their priority.
To allow the exploration the space of the possible solutions, the algorithm perturbs both the priorities and the desired departure times of the train families. At the end of each internal iteration the priority of the families is varied randomly, so that at the next iteration the sequence in which the different families are scheduled may change. The internal cycle is iterated times for every iteration of the external cycle.

The output of the TTPSW is a complete timetable for each train of the families in C that TTPSW has been able to schedule. Specifically, the software reports the arrival time, the departure time, the platform and the route of the train in each station, junction or halt that the train visits along its line.

5 Estimating robustness

A timetable created by the TTPSW is normally feasible, but it appears important to verify its robustness, at least on its core sections. Microscopic simulation represents the most accurate tool for reproducing railway operations including their stochastic components. Simulators such as OpenTrack [8] can reproduce most processes involved in rail traffic and comprehend not only its deterministic aspects, but also human factors. This is particularly relevant in order to simulate traffic under realistic conditions, considering variability at border, various driving styles and stop times [9].
In the approach presented in this paper, stochastic microscopic simulation is used to estimate the delay measures that would be obtained operating the timetable obtained using the TTPSW on the infrastructure defined previously and under given variability. A combination of mean delay, punctuality and of the Frequency of Delay Index is used to evaluate the quality of operations. Should a timetable not prove robust, the planner can decide to add buffer times which are a characteristic of each train group or release other timetable constraints and then run the TTPSW again.

6 Case study: Norway

The Norwegian Rail Administration JBV manages about 4100 km of lines, mostly single track. It is pursuing an ambitious improvement plan that will gradually improve the capacity of the network, especially on the most-densely used lines. the core of this project will be the new 20 km-long tunnel of the Follo line (Follobanen), a 22.5 km long double-track line built for 250 km/h: it will triple the available capacity on the saturated section of the Ostfold line between Oslo and Ski allowing more frequent and faster services also on the branch lines. Selected sections of the long-distance corridor connecting Oslo with Trondheim will be doubled, as well as a part of the Ostfold and Vestfold lines; besides this major improvements, a series of smaller measures will allow operating the target volume of services on each line.
JBV started a project, called R2023, to select and define the key requirements of the investments effectively required to reach the target capacity and robustness of the operations by at-the-same avoiding unnecessary expenses. The project is an ideal case study for the approach, since different scenarios of infrastructure timetable have to be created to allow selecting the best combination of improvements, by at-the-same-time considering the constraints of rail operations. The entire network was modelled using the mesoscocpic model, obtaining very satisfying results especially in the estimation of the running and blocking times to be used as input for the TTPSW: they are calculated in less that 5’ on a standard desktop PC.
The usability of the TTPSW proved less satisfying: multiple testes were required to estimate a set of parameters – in both absolute and relative terms – that lead to a realistic timetable structure and infrastructure utilization. Coherently with the operating principle of the algorithm, an inappropriate set of parameters results in a timetable with too long running time margins at some trains or with some missing trains and not in long computational times.
The algorithm is currently able to create a timetabel for a significant part of the network in less than 10 minutes, although it is not always able the same number of trains as a skilled planner. To cope with this weakness, the TTPSW was «guided» adding or releasing contraints until all expected services were scheduled. Further tests are currently being performed, especially in order to identify the sets of parameters and the number of iterations that most frequently lead to satisfying results.

7 Conclusions and outlook

To gain competitiveness in a rapidly changing economic context, railways need efficient and effective improvements, which allow facing strength market conditions. The impact of such improvements has to be precisely evaluated, to allow choosing interventions and combining them in long-term development programs.
The presented methodology allows a realistic and efficient planning of railway timetables and networks, creating several alternatives that can be compared easily.
The first large-scale application is showing promising results, with very satisfying results especially in terms of computation times. The tests show that the model is able to compute realistic solutions in a few minutes: the position of the slots on the timetable graph appears similar to that used in the timetable created by practitioners.
The promising results obtained in the first large-scale tests foster further improvements to the algorithm and the whole approach, in order to make it more reliable and understandable for practitioners, especially concerning the selection of the constraints and parameters.
The method used to estimate mean blocking times on line sections will be improved, while its effective accuracy will be estimated, especially considering longer distances between stations. Results obtained in the most used part of the network will be compared to micro-simulation in order to estimate the difference between micro- and mesoscopic models in terms of blocking times and conflicts effectively considered.
While all mentioned improvements appear relevant in order to benchmark the quality of the results their applicability to very large networks, the most extensive tests will be carried out in order to find some general rules that allow defining the parameters and constraints that lead to a realistic timetable structure.

References

[1] Hansen, I. A.: “State-of-the-art of railway operations research,” in Computers in Railways X (J. Allan, R. Hill, C. Brebbia, G. Sciutto, and S. Sone, eds.), pp. 565–577, WIT Press, Southampton, UK, 2006.

[2] de Fabris, S., Medeossi, G., Longo, G., Pesenti, R.: “Automatic generation of railway timetables based on a mesoscopic infrastructure model” to appear in Journal of Rail Transport Planning & Management (2014)

[3] Caimi, G., Laumanns, M., Schüpbach, K., Wörner, S., Fuchsberger, M.: “The periodic service intention as a conceptual framework for generating timetables with partial periodicity,” Transportation Planning and Technology, vol. 34, no. 4, pp. 323–339, 2011.

[4] Kroon, L., Huisman, D., Abbink, E., Fioole, P.-J., Fischetti, M., G. Maróti, G., Schrijver, A., Steenbeek, A., Ybema, R.: “The new Dutch timetable: The OR revolution,” Interfaces, vol. 39, no. 1, pp. 6–17, 2009.

[5] Schlechte, T., Berndorfer,R., Erol, B., Graffagnino, T., Swarat, E.: Micro-macro transformation of railway networks. Journal of Rail Transport Planning & Management, 1, pp. 38–48, 2011.

[6] Caimi, G.: Algorithmic decision support for train scheduling in a large and highly utilised railway network, ETH Zurich, PhD Thesis, 2009.

[7] Huerlimann, D.: “Object-oriented modelling in railways”; ETH Dissertation Nr. 14281; (in German), 2001.

[8] Longo, G., Medeossi, G.: Capacity vs Reliability in railways: A stochastic micro-simulation approach, in Road and Rail Infrastructure II (S. Lakusic, ed.), University of Zagreb, Zagreb, pp. 97-104, 2012.

[9] Medeossi, G.: “Capacity and Reliabiiity on railway networks: a simulative approach”, PhD Thesis, University of Trieste

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MICROSCOPIC SIMULATION OF RAILWAY OPERATION FOR DEVELOPING INTEGRATED TIMETABLES

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Abstract

OpenTrack is a user-friendly railroad network simulation program developed at the Swiss Federal Institute of Technology's Institute for Transportation Planning and Systems (ETH IVT). It is a microscopic model that simulates rail system operations based on user defined train, infrastructure, and timetable databases. OpenTrack functions as a railroad laboratory, by, for example, allowing users to define incidents and take infrastructure out of service to evaluate alternative scenarios. The program uses a mixed discrete/continuous simulation process that calculates both the continuous solution of train motion equations and the discrete processes of signal box states and delay distributions. It generates a wide variety data that can be easily presented in many formats including graphs (e.g. time-space diagrams), tables, and images. OpenTrack's main uses have been to evaluate and test infrastructure plans and operating schedules to optimize network and timetable design. It can be run on several different computer platforms and incorporates the benefits of object oriented programming language with a common data interface structure.

Keywords: railway simulation, railway planning, railway operations analysis, RailML, XML, object oriented modelling

1 Introduction

At its most basic level a rail simulation program is a set of simplifying mathematical assumptions that attempt to duplicate actual train operations. It allows users to evaluate the impacts of user-defined changes on the rail system operation. There are many types of railroad simulation programs; each of which is designed to answer a different type of question. For example there are models designed to help dispatchers decide how to route trains through a network in real time. Similarly there are models designed to help planners identify infrastructure requirements for planned schedules ten years in the future. As with most planning processes, a critical component of railroad planning is choosing the best computer simulation tool to use in any given situation. Rail simulation models can be categorized as macroscopic or microscopic similar to other transportation models. Both types are outlined below.

1.1 Macroscopic Simulation Models

Macroscopic models use average or other statistical data to evaluate operation of the transportation system; they do not model individual unit (e.g. train) operations nor do they consider how trains are impacted by other trains or how safety systems impact train performance. A common type of macroscopic model is the NEMO model developed at the University of Hannover.

1.2 Microscopic Simulation Models

Microscopic simulation models attempt to replicate the actual operation of a railroad over time. They do this by modeling the operation of each individual train during a user-defined time step (often one second) and then repeating the process for the entire simulation period. Microscopic models consider the impact of trains on each other when they simulate train operations. For example, if during a particular time step, a train occupies a block that a second train wishes to occupy, then the program will model the second train as performing according to the (user defined) safety system parameters. In the case of a simple block signal system the second train would stop at the block entry signal and wait until the first train clears the block before proceeding.
There are two types of microscopic models: synchronous and asynchronous. Synchronous models simulate all train operations in a single model run while asynchronous models simulate operations in a series of model runs.
In an asynchronous simulation the highest priority trains are modeled in the first run, then this schedule is locked and the second set of trains is modeled; the operation of the second set of trains does not impact the first set of trains, but is impacted by the first set (similarly, the operation of the third set of trains does not impact the first or second sets, and so forth). These types of models are often used for timetable construction since they can replicate ideal operational planning. A good example of this type of program is STRESI developed by RWTH in Aachen.
In contrast, synchronous models simulate all the trains operating in the modeled network at the same time, thus they provide a good way to simulate realistic operating situations (e.g. the impact of train delays on network operations). These models enable users to specify rules that the program uses to make dispatching decisions when there are conflicts between trains. These rules include simple train priorities (e.g. passenger trains before freight trains) as well as more complex rules designed to optimize some particular function (e.g. dynamic overtaking). In essence these models attempt to replicate good dispatching decisions.
OpenTrack is a microscopic synchronous railroad simulation model. It provides users with a great deal of flexibility for defining different dispatching logic as well as operational variables in a user-friendly manner.

2 OpenTrack Rail Simulation Program

OpenTrack was developed at the Swiss Federal Institute of Technology's Institute for Transportation Planning and Systems (ETH IVT). The project's goal was development of a user-friendly railroad simulation program that can run on different computer platforms and can answer many different questions about railway operations. Figure 1 illustrates the three main elements of OpenTrack: data input, simulation, and output.
OpenTrack is a microscopic synchronous railroad simulation model. As such it simulates the behaviour of all railway elements (infrastructure network, rolling stock, and timetable) as well as all the processes between them. It can be easily used for many different types of projects including testing the stability of a new timetable, evaluating the benefits of different long-term infrastructure improvement programs, and analyzing the impacts of different rolling stock.

2.1 Input Data

OpenTrack administers input data in three modules: rolling stock (trains), infrastructure, and timetable. Users enter input information into these modules and OpenTrack stores it in a database structure. Once data has been entered into the program, it can be used in many different simulation projects. For example, once a certain locomotive type has been entered into the database, that locomotive can be used in any simulation performed with OpenTrack. Similarly, different segments of the infrastructure network can be entered separately into the database and then used individually to model operations on the particular segment or together to model larger networks.

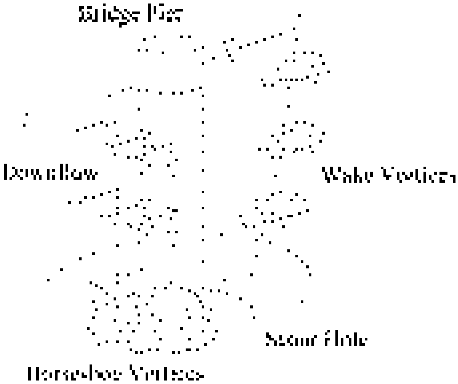

Figure 1 Data flow during a simulation project

Train data (locomotive and wagons) is entered into the OpenTrack database with easy to use forms displayed using pull down menus. Infrastructure data (e.g. track layout, signal type/location) is entered with a user-friendly graphical interface; quantitative infrastructure data (e.g. elevation) is added using input forms linked to the graphical elements. Following completion of the RailML data structure for rolling stock and infrastructure, OpenTrack will be modified to enable train and infrastructure data to be directly imported from RailML data files.
Timetable data is entered into the OpenTrack database using forms. These forms include shortcuts that enable data input to be completed efficiently. For example, users can designate hourly trains that follow the same station stopping pattern an hour later. Since OpenTrack uses the RailML structure for timetable data, timetable data can also be entered directly from various different program output files as well as database files.
One advantage of OpenTrack is that it enables users to adjust many variables that impact railroad operations. For example, users can simulate the impact of weather on traction by specifying the adhesion scenario (good, normal, bad). OpenTrack then estimates locomotive traction power using a percentage (also user-defined) of that calculated using the Curtius and Kniffler formula. While OpenTrack provides standard default values for all variables, having the ability to adjust variables makes the program quite useful.

2.2 OpenTrack Simulation Process

In order to run a simulation using OpenTrack the user specifies the trains, infrastructure and timetable to be modeled along with a series of simulation parameters (e.g. animation formats) on a preferences window. During the simulation, OpenTrack attempts to meet the user-defined timetable on the specified infrastructure network based on the train characteristics.
OpenTrack uses a mixed continuous/discrete simulation process that allows a time driven running of all the continuous and discrete processes (of both the vehicles and the safety systems) under the conditions of the integrated dispatching rules.
The continuous simulation is dynamic calculation of train movements based on Newton's motion formulas. For each time step, the maximum force between the locomotive's wheels and the tracks is calculated and then used to calculate acceleration. Next, the acceleration function is integrated to provide the train's speed function and is integrated a second time to provide the train's position function.

The discrete simulation process models operation of the safety systems; in other words, train movements are governed by the track network's signals. Therefore, parameters including occupied track sections, signal switching times, and restrictive signal states all influence the train performance. OpenTrack supports traditional multi-aspect signaling systems as well as new moving block train control systems (e.g. European Train Control System – ETCS signaling).

2.3 Dynamic Rail Simulation

OpenTrack is a dynamic rail simulation program. As such, the simulated operation of trains depends on the state of the system at each step in the process as well as the original user-defined objective data (e.g. desired schedule).
A simple way of describing dynamic rail simulation is that the program decides what routes trains use while the program is running. For example, when building the network, users identify various different routes that trains can use between two points; OpenTrack decides, during the simulation, which route the train will use by assigning the train the highest priority route available. If the first priority is not available, OpenTrack will assign the train the second highest priority route and so on.
OpenTrack's dynamic nature allows users to assign certain attributes to specified times in the simulation. Thus, users can assign a delay to a particular train at a given station and time, rather than being limited to assigning a delay at the start and using it through the entire simulation. Similarly, users can define other types of incidents (e.g. infrastructure failures, rolling stock breakdowns) for particular times and places.
Finally, dynamic simulation enables users to run OpenTrack in a step-by-step process and monitor results at each step. Users can also specify exactly what results are displayed on the screen. Running OpenTrack in a step-by-step mode with real time data presented on screen helps users to identify problems and develop alternative solutions.

2.4 OpenTrack Output

One of the major benefits of using an object oriented language is the great variety of data types, presentation formats, and specifications that are available to the user. During the OpenTrack simulation each train feeds a virtual tachograph (output database), which stores data such as acceleration, speed, and distance covered. Storing the data in this way allows users to perform various different evaluations after the simulation has been completed.
OpenTrack allows users to present output data in many different formats including various forms of graphs (e.g. time-space diagrams), tables, and images. Similarly, users can choose to model the entire network or selected parts, depending on their needs. Output can be used either to document a particular simulation scenario or as an interim product designed to help users identify input modifications for another model run.

3 Application of OpenTrack at NeusiedlerSeeBahn (NSB)

The NeusiedlerSeeBahn is responsible for the line from Neusiedl am See to Pamhagen. At Neusiedl am See the branch line from/to Eisenstadt is located. Pamhagen is the last station in Austria before the Hungarian border. From Neusiedl am See a line leads to Parndorf Ort where there is the connection to the Eastern line of Austrian Railways which is connecting Vienna and Budapest. People living at the villages located between Neusiedl am See and Pamhagen typically go for work to Vienna every single day of the week. Therefore the NSB is interested in offering shorter traveling times to Vienna. To achieve this goal some investments were done in the last years to increase the track speed. OpenTrack has been successfully used to evaluate the shortenings of running times for local trains and regional trains.

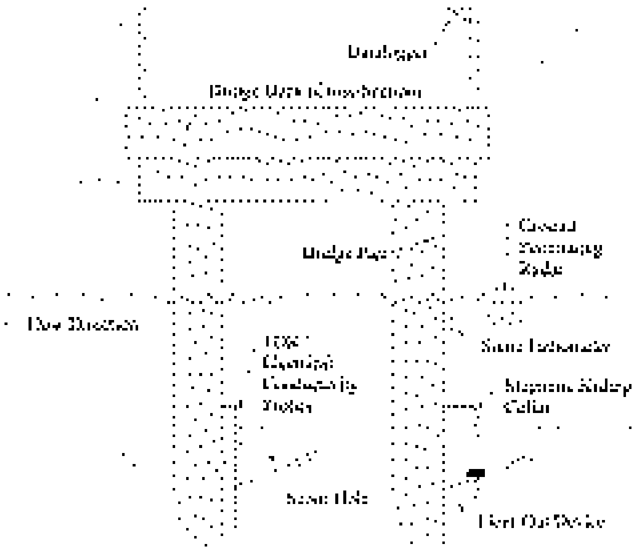

Figure 2 Infrastructure layout of the line from Neusiedl am See to Pamhagen

Figure 2 shows the topology of the line from Neusiedl am See to Pamhagen. Crossing opportunities are located at Bad Neusiedl, Mönchhof-Halbturn, Frauenkirchen, St. Andrä am Zicksee, Wallern and Pamhagen. The first step in the project was the check of the model with the existing timetable. This step allowed the exact calculation of running times reserves included in the existing timetable.

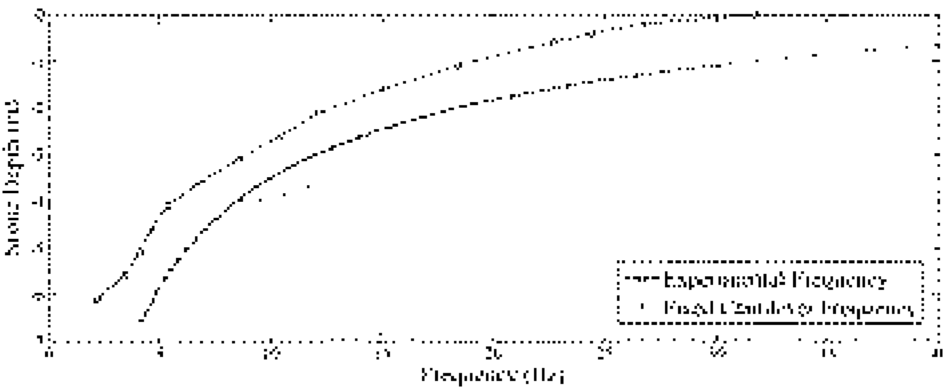

Figure 3 Simulation result of the existing timetable

Figure 3 shows the result from the simulation of the existing timetable. It has to be noted that one course during the day is only going to St. Andrä am Zicksee and not to the terminal station at Pamhagen. Additionally there is no service during the morning hours towards the region between Neusiedl am See and Pamhagen. Exactly the same happens during the evening when there is no service towards Vienna. The reason for this asymmetry in the timetable can be only explained by the saving of operational costs. For pupils there are two additional services during the day to allow them traveling to and back from school. Unfortunately the infrastructure does not allow a crossing between Bad Neusiedl and Mönchhof-Halbturn which would be required to run a 30 minutes interval in both directions. Furthermore the infrastructure model had to be extended with the upgraded track speed limits which had been indicated by markers at the related vertices. Due to the increase of the track speed the crossing shifts from St. Andrä am Zicksee towards Wallern because of keeping the arrival and departure times at Neusiedl am See (see Figure 4).

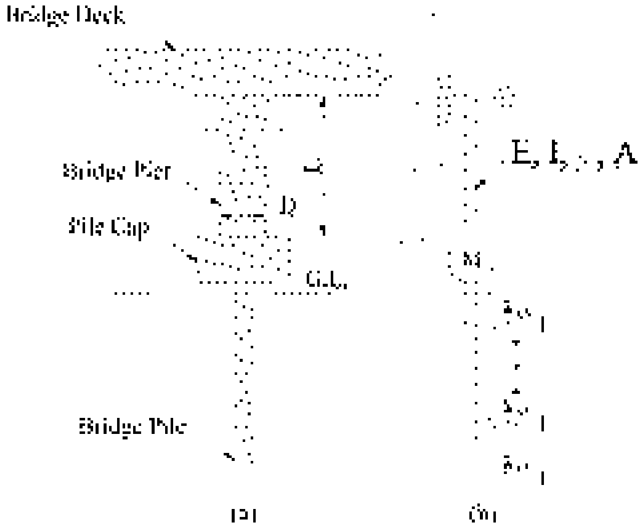

Figure 4 Result of the simulation for upgraded infrastructure

Local and regional trains benefit from the increase of track speed by a shortening of running times of 5 minutes each. Local trains which stop at every single station run in 33 instead of 38 minutes and regional trains run in 28 instead of 33 minutes. A shortening of 5 minutes in terms of running time could be also achieved by introducing today regional trains. Of course the increase of the track speed in combination with the introduction of regional trains leads to an overall reduction of running time of 10 minutes for regional trains. An open point for further investigations is the integration of both services at the same time because each crossing of two trains will lead to an increase of running time while one train has to take the siding track in a crossing station. This could require the demand to upgrade also the frequently used switches to allow higher speeds for the siding track.

4 Conclusions

OpenTrack is an efficient and effective railroad simulation program. It has been successfully used in many different railway planning projects throughout the world. The program's use of object oriented programming and the RailML data structure makes it particularly effective since the program can be modified relatively easily to address specific applications and since data can be transferred easily to and from other programs based on RailML.

References

[1] Huerlimann, D.: Object oriented modeling in railways; ETH Dissertation Nr. 14281; 2001 (in German).

[2] Huerlimann, D., Nash, A.: OpenTrack – Simulation of Railway Networks, User Manual Version 1.6; OpenTrack Railway Technology GmbH; Zürich, Switzerland; 2010; 160 pages.

[3] For more information on OpenTrack see the project website: www.opentrack.ch

9 TRAFFIC SAFETY

RELATION BETWEEN SPEED INCONSISTENCY AND DRIVING SAFETY ON CROATIAN STATE ROAD D-1

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Abstract

Traffic accidents and road fatalities present a serious problem of the modern age. Statistics show that large percentage of traffic accidents with fatalities occur on horizontal curves which indicates that, in addition to drivers' errors, causes of road accidents can be found in the characteristics of the road alignment. Driving safety depends on several factors and one of the main causes of accident occurrence is the lack of geometric design consistency in terms of maintaining the desired travel speed. Design consistency refers to the ability of geometric characteristics of the road to conform to driver expectancy. A consistent road design ensures coordinated successive elements producing harmonized driver behaviour with no surprising events. Beside the successive elements consistency, a good road design must establish the balance of superelevation and side friction values in curves with actual driving speeds.
Although numerous speed studies have been conducted, most of them were based on spot speeds and certain assumptions providing incomplete information about the actual speeds. The limitations of the existing studies indicate the need for more detailed research of actual driver behaviour and the inclusion of real speeds in the road design procedure. This paper presents the free flow speed analysis, with speeds recorded on a segment of the state road D1 in Dalmatia region of Croatia using an innovative GPS based methodology. This methodology allows the continuous speed data collection and gives an accurate picture of drivers' behaviour. The data were collected on 27 km long road segment and used for three types of analysis: relationship between project speeds and actual speeds of most drivers, comparison between side friction factors (demand, supply and design) and creation of the operating speed model in horizontal curves.

Keywords: *GPS, operating speed, project speed, horizontal curve, side friction factor, operating speed model*

1 Introduction

Factors mainly affecting road safety are: the driver, the vehicle and the road [1]. Previous road safety studies showed that accidents often occur on horizontal curves which indicates that there is a relationship between the traffic safety and road alignment characteristics. Previous accident researches have found that accident rates on horizontal curves are 2 to 5 times higher than the accident rates on tangents [2, 3]. Also in Croatia a large percentage of fatalities are related to horizontal curves. According to [4], approximately 40% of total fatalities that occurred in crashes in 2011 in Croatia occurred along horizontal curves.

2 Road design practice

All current geometric design policies are based on the design speed concept. A road design procedure begins with selecting a speed called "design speed" which is used for determination of the various geometric design features of the roadway. Among other elements, the most important one is determination of the allowable horizontal curve radii. The use of design speed enables driving safety in those sharpest curves, while road segments with flatter curves allow for higher driving speeds [5]. Some road design guidelines, like German [6] and Australian [7], recognize this by using the 85^{th} percentile operating speed obtained by field survey. The operating speed is defined as the speed below which 85% of vehicles actually drive under free flow conditions.
Croatian guidelines for the road design [8] use theoretical value of speed, called project speed (V_p), rather than the 85th percentile operating speed. Project speed is defined as the maximum expected speed in free flow conditions that can be achieved with sufficient safety on a particular part of the road segment depending on its horizontal and vertical characteristics. It is determined from the basic equation of vehicle stability in horizontal curves, as a function of applied curve radius or largest applied longitudinal grade (smaller value is chosen):

$$V_p = \sqrt{127 \cdot R \cdot (e_{max} + f_{Rdesign})} \tag{1}$$

where:

V_p project speed (km/h);
R curve radius (m);
e_{max} maximum permissible superelevation;
$f_{Rdesign}$ design side friction factor.

Croatian guidelines define the project speed as a criterion for determining superelevation and stopping sight distance. They also provide consistency criteria in terms of design and project speed consistency and consistency of project speeds within one road section. The values of project speeds should not exceed the design speeds by more than 20 km/h, otherwise the guidelines require that either the project speed be increased or the road alignment be modified to reduce the project speed. Also, the maximum differences in project speeds within one section should be less than 15 km/h.
Driving speeds are influenced by geometric elements such as horizontal curvature, cross-sections and grade, so it should be possible to control speed throughout the appropriate selection of geometric design standards for these elements [9]. Designing the geometric elements based on speeds not harmonized with actual speeds can result in inappropriate road elements. In particular, inadequate speeds in the road design process could result in superelevation and sight distance values that are less than optimal for safe and comfortable ride.

3 Data collection

Although numerous studies have been developed in order to determine operating speeds at curves, most of them were based on spot speeds and certain assumptions about drivers' behaviour. The lowest speeds along the horizontal curves and highest speeds along tangents are considered to be the desired drivers' speeds. The researchers collected speed data at specific locations of a roadway, mostly at the middle point of horizontal curves, using radar gun or similar device. Due to the lack of data, many models used the assumption of a constant velocity along a horizontal curve and the assumption that the acceleration and deceleration occur only on tangents. These assumptions may not be realistic [10, 11]. Except the unrealistic assumptions of driver behaviour, there are also some other disadvantages of spot speed

data measuring, like cosine error, drivers changing their behaviour in the presence of test equipment and human error when reading data from the device display.
Because of the many shortcomings of the spot speeds and with the development of technology, more and more authors focus on continuous data measuring. In the past decade, several operating speed studies have been conducted based on continuous measured data using a GPS device [1, 10, 12, 13]. This paper presents the operating speed prediction model for horizontal curves based on the speed data from 10 km long road segment. The data were collected by using an innovative GPS device Performance Box, a high performance 10HZ GPS engine, which measures speed, position, acceleration and heading ten times a second. Performance Box uses a GPS system with an update rate of 10 samples per second, which is faster GPS engine than the devices of the previous reserches which were working with frequency of 1HZ. The GPS data collection methodology allows the determination of individual minimum speeds on curves and maximum speeds on tangents (location and value), unlike the spot speed models that used unrealistic assumptions of driver behaviour. Beside the speeds, GPS technology enables the determination of drivers' path radii of the curves, on which the side friction demand depends.

Figure 1 Analyzed segment of the state road D1

The test rides were carried on a road segment which consists of horizontal curves with radii varying from 80 to 1000 m. Operating speed prediction model for horizontal curves was determined based on the speed data from 10 km long road segment, and the model validation was made with the data from 1 km long segment of the road (Fig. 1). Analyzed road segment is the two-lane state road with relatively low traffic volume (according to [14], the average annual daily traffic was 1377 veh/day in 2011). The rides were recorded during day under optimal weather conditions and free flow conditions, in order to reduce the conditions not related to the geometry of the alignment. Test driver sample consisted of 15 people with passenger cars of different types and ages.
In this study, the values of speeds and radii relevant for analysis, V_{85} and R_{15}, are determined based on data collected from 15 individual drivers [15]. The 85th percentile speed V_{85} is the speed below which 85% of drivers actually drive. The path radii of 15% of drivers are smaller than the 15th percentile radius R_{15}.

4 Analysis and results

Although Croatian guidelines for the road design use the term “operating speed”, it is actually represented as theoretical value named project speed. The question is how much are the values of project speeds consistent with real speeds of most drivers and how the use of project speed affects the side friction demand. Operating speeds determined from the data collected on 10 km long road segment were compared to the project speeds, the comparison between side friction values (demand, supply and design) was made and the operating speed prediction model for horizontal curves was determined.

4.1 Disparity between project and operating speed

The analyzed segment of the road D1 has been designed as a 2nd category state road with design speed of 60 km/h. The values of project speeds in horizontal curves were determined according to Croatian guidelines, for a 2nd category road and actual radii. The actual curve radii on horizontal curves were determined from the road surveying which was made in 2006 by Hrvatske ceste (company for managing, constructing and maintaining of state roads in Croatia).

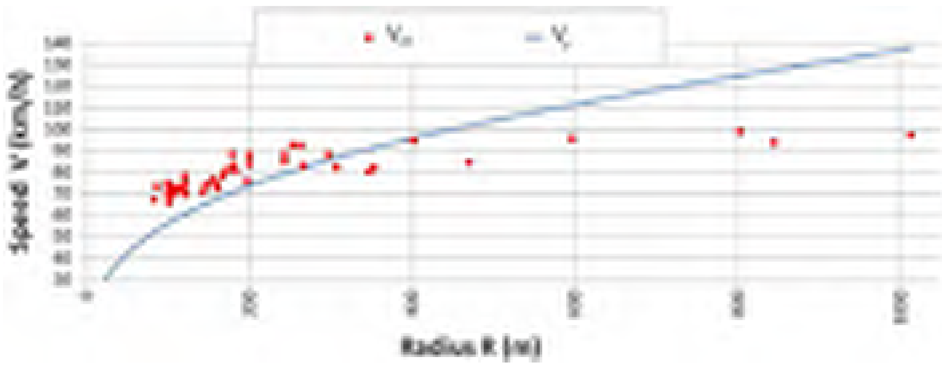

Figure 2 Operating and project speeds in horizontal curves

The comparison between the operating and project speeds is shown in Fig. 2. It can be seen that the operating speed is greater than the project speed in all curves with radii smaller than about 300 m. The project speed is lower than the operating speed which results in insufficient values of stopping sight distances and superelevations optimal for a comfortable ride. Very important elements of the road from a safety point of view are determined based on speeds much lower than actual speeds which may be the cause of accident.

4.2 Side friction demand, supply and design

Differences between operating and project speeds result in exceeding the limiting values of side friction. The side friction demand factors (f_{Rd}) were calculated for actual superelevations in horizontal curves and relevant radii and speeds (V_{85} and R_{15}). The actual radii and superelevations on horizontal curves were determined from the road surveying made by Hrvatske ceste. The side friction demands are compared to the side friction supply (f_{Rmax}) and design side friction factors ($f_{Rdesign}$). The comparison is presented in Fig. 3.

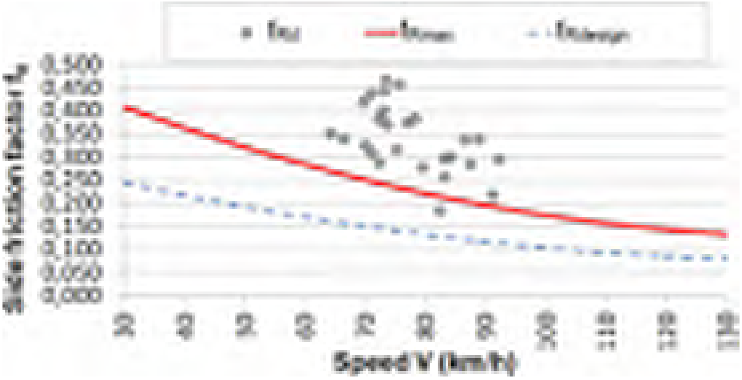

Figure 3 Relationship between side friction demand, supply and design factors

The side friction demands exceed design side friction factors in all curves and they are higher than the peak side friction supply values in almost all curves. This point to the danger of skidding, especially in poor driving conditions such as wet and dirty pavement and worn tires.

4.3 Operating speed model for horizontal curves

The majority of existing operating speed models were made using linear regression with variety of geometric and traffic characteristics of the alignment [11]. The main independent variables chosen for use were radius of the curve and approach speed (speed at which vehicle approaches a curve). This paper describes a multiple regression procedure to determine horizontal curve speed model based on data collected on 10 km long segment of state road D1, with horizontal curve radius (R) and the approach speed ($V_{approach}$) as independent variables [15]. The model validation was made with the data from 1 km long segment of the road. Data from 44 horizontal curves (80 m ‹ R ‹ 1000 m) were used for model determination and from 4 curves (170 m ‹ R ‹ 300 m) were used for model validation. Based on the collected data and using a multiple linear regression, the following model was developed:

$$\widehat{V}_{85} = 0{,}023 \cdot R + 0{,}471 \cdot V_{approach85} + 35{,}443 \tag{2}$$

The coefficient of determination of the model is equal to 0,761. The accuracy of the model was also expressed by the mean absolute percentage error, defined by the formula:

$$MAPE = \left| \frac{\widehat{V}_{85} - V_{85}}{V_{85}} \right| \cdot 100\% \tag{3}$$

The overall mean absolute percentage error (MAPE) for the data from 10 km long segment is 4,2% and the maximum MAPE is 17,2%. Operating speeds derived from the collected data and model predicted speeds in horizontal curves are shown in Fig. 4. It should be noted that the model fits the data well.

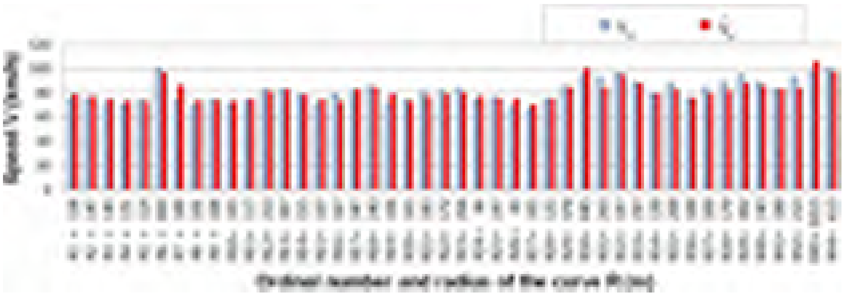

Figure 4 Operating and model predicted speeds in horizontal curves

Figure 5 The values of mean absolute percentage error for the model data and validation data

The validation of the speed prediction equation for horizontal curves (2) was performed by comparing the model predicted speeds to field observed speeds from the 1 km long segment of the road. The MAPE value for the validation data is 4,6% and the maximum value is 6,3%. The MAPE values for the data from 10 km long segment (model data) were compared to the values for the data from 1 km long segment of the road (validation data), which is presented in Fig. 5. The MAPE values for validation data are similar to the MAPE values for model data what means that, according to radii and approach speeds, the model predicts accurately the curve speed choice of the driving population.

5 Conclusions

The main factor in the road design procedure is the speed of the vehicle. The relevant speed should be determined primarily based on safety and driving comfort. Speed studies in many countries have found that speeds used in the road design underestimate the actual speeds of most drivers. Most of European countries use operating speed to determine the values of superelevations in curves and stopping sight distances and for the evaluation of speed consistency. The main difference is in the way in which the operating speed is defined and determined. According to Croatian guidelines, the operating speed is a theoretical value defined as project speed. This speed is mostly lower than the 85th percentile speed obtained by field survey which leads to the road elements inconsistent with the actual speeds. That may be a potential cause of an accident. This paper presented an experimental investigation of the drivers' speed behaviour on horizontal curves of two-lane state roads in Croatia. It was found that the operating speed is greater than the project speed in curves with radii smaller than 300 m. Differences between operating and project speeds result in exceeding design side friction factors and peak side friction supply values. These findings indicate the need for changes in the existing road design, mostly in terms of harmonization the relevant speeds that are used in the road design with actual speeds of most drivers.

It is also presented an example of determining the operating speed prediction model for horizontal curves, based on the collected data and using a multiple linear regression. The model was calibrated with continuous speed data measured by an innovative GPS device. Unlike previous spot speed methodologies, GPS allows the determination of locations and values of critical speeds on tangents and horizontal curves. Most previous speed models were based on the assumption that the speed remains constant throughout a curve and that drivers reach their highest speeds in the middle of tangents and reach their lowest speeds in the middle of horizontal curves. The continuous data collected in this study showed that these assumptions, in general are not realistic. The speed throughout a curve doesn't remain constant and locations of the lowest speeds on curves and highest speeds on tangents differ from driver to driver. Although the operating speed model is obtained with a relatively high coefficient of determination and relatively low values of mean absolute percentage errors, it is necessary to include additional independent variables and field data, to make the model more reliable.

References

[1] Pérez Zuriaga, A.M.: Characterisation and Modelling of Operating Speed on Conventional Roads Using Naturalistic Observation of Light Vehicles, Universitat Politècnica de València, Department of Transport Infrastructure and Engineering, doctoral thesis, 2012

[2] Lamm, R., Choueiri, E.M., Mailaender, T.: Accident Rates on Curves as Influenced by Highway Design Elements – an International Review and an In-Depth Study, Proceedings, Road safety in Europe, Gothenburg, Sweden, Swedish Road and Traffic Research Institute, Linkoeping, Sweden, VTI Report 344A, pp.33-54, 1989

[3] Fink, K.L., Krammes, R.A.: Tangent Length and Sight Distance Effects on Accident Rates at Horizontal Curves on Rural Two-Lane Highways, Paper presented at Transportation Research Board, 74th Annual Meeting, Washington D.C., 1995

[4] Republic of Croatia, the Ministry of the Interior, Secretariat of the Ministry - Department for Strategic Planning, Analysis and Development: Bilten o sigurnosti cestovnog prometa 2011., Zagreb, 2011

[5] Cvitanić, D., Vukoje, B., Breški, D.: Methodes for Ensuring Consistency of Horizontal Alignment Elements, Građevinar 64, pp. 385-393, 2012

[6] Forschungsgesellschaft für Straßen - und Verkehrswesen, Arbeitsgruppe "Straßenentwurf": Die Richtlinien für die Anlage von Straßen – Teil: Linienführung (kurz RAS-L), Germany, 1973, 1984, 1995

[7] Austroroads: Rural Road Design: Guide to the Geometric Design of Rural Roads, Sydney, Australia, 1997

[8] Ministarstvo pomorstva, prometa i veza, Republika Hrvatska: Pravilnik o osnovnim uvjetima kojima javne ceste izvan naselja i njihovi elementi moraju udovoljavati sa stajališta sigurnosti prometa (NN 110/01), Zagreb, 2001

[9] Safety, Speed and Speed Management: A Canadian Review, Transport Canada 1997

[10] Wang, J.: Operating Speed Models for Low Speed Urban Environments Based on In-Vehicle GPS Data, Dissertation, Georgia Institute of Technology, 2006

[11] Hassan, Y., Sarham, M., Dimaiuta, M., Donnell, E., Himes, S., Porter, R., Psarianos, B., García, A., Perco, P. y Taylor, M.: Modeling Operating Speed: Synthesis Report, Transportation Research Circular number E-C151, Transportation Research Board, 2011

[12] Jiang, Y., Li, S.: Measuring and Analyzing Vehicle Position and Speed Data at Work Zones Using Global Positioning System, Institute of Transportation Engineers, ITE Journal, 2002

[13] Cafiso, S., Cerni, G.: A New Approach to Define Continuous Speed Profile Models for Two Lane Rural Roads, 91st Annual Meeting Transportation Research Board, 2011

[14] Hrvatske ceste d.o.o.: Brojenje prometa na cestama Republike Hrvatske godine 2011. – sažetak, Zagreb, 2011

[15] Proso, A.: Free Ride Dependence on Geometric Characteristics of the Road Route, diploma thesis, 2013

THE NEED FOR SAFER AND FORGIVING ROADS

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Abstract

Improving road safety has been the key objective for road authorities worldwide for the last years. Lately, many concepts were adopted to reduce the number of fatalities, concepts like self-explaining roads, low cost measures or forgiving roads. As new research findings are published, differing theories evolve and road safety visions change. Nowadays, around 30% of accidents on the entire EU road network are caused by inadequate infrastructure. The way roads are laid out and designed can reduce the exposure to traffic of vulnerable road users, reduce the probability that crash and injury occur when these users are exposed and reduce the severity of injury if it occurs. Substantial and sustainable casualty reductions can be achieved in relatively short time and at relatively short cost by identifying and treating high risk infrastructure sites, creating safer and forgiving roads. The aim of this paper is to improve traffic safety by increasing the awareness of road authorities, in order for them to implement road safety measures following the concepts of forgiving roadsides and taking into consideration the human factors also.

Keywords: road safety, road elements, forgiving roads, traffic accidents

1 Introduction

More than 1.2 million people are killed on the road every year and more than 20 million are injured, according to a World Health Organization recent report. Despite the fact that 10% of the total accidents are single vehicle accidents (typically run-off-road accidents) the rate of these events increase up to 45% when only fatal accidents are considered, [1]. One of the key issues of this dramatic increase for the high rate of this type of accidents is the lack of forgiving roadsides. The forgivingness of the environment and of road users is defined as injury limitation through a forgiving road environment and in the meantime, an anticipation of road user behavior.

A forgiving road is defined as a road that is designed and built in such a way as to interfere with or block the development of driving errors, but also to avoid or mitigate negative consequences of driving errors, allowing the driver to regain control and either stop or return to the travel lane without injury or damage, [2].

A large number of research studies have been conducted in the past years, studies which contributed to the development of the road design standards for improving roadside design. They suggest [3] that the stages in any strategy for improving the siting and design of street equipment can be further developed and extended as follows:

Table 1 Main principles for forgiving roads.

Existing roads	Designed roads
Eliminating unnecessary obstacles	Designing roads without any dangerous street equipment problems
Moving obstacles further away from the roadside	Designing a clear zone at the side of the road
Modifying the structure of the obstacles	Designing street equipment to be more forgiving
Isolating certain obstacles with new and improved types of safety device	Protecting street equipment with a barrier to absorb some of the energy of the impact

To develop a forgiving road environment, certain characteristics must be included and measures should be taken, considering standard road safety measures, but also a practical tool for assessing the effectiveness of a roadside treatment, as can be seen further in this paper.

2 Standard road safety measures for forgiving roads

2.1 Roadside clear zones

A very frequent cause of traffic accidents is the existence of obstacles along the road, obstacles which don't forgive drivers mistakes. These walls, which were inadequate designed and built, modify the incidence of crashes. Their presence has two major implications regarding road safety: the danger of collision and the obstruction of visibility, Figure 1.

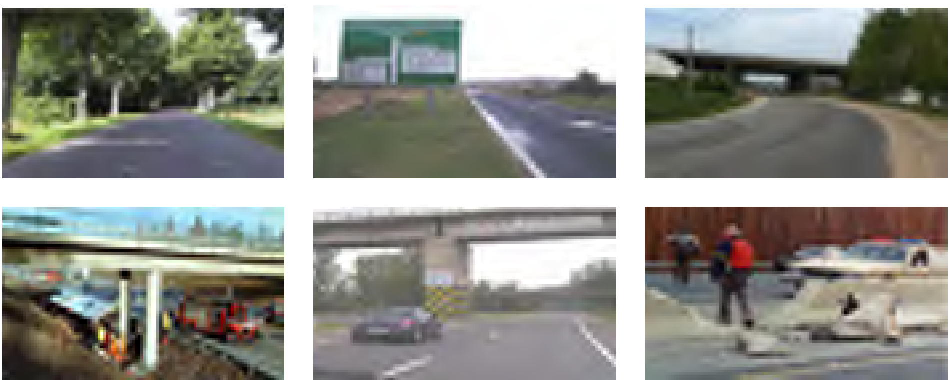

Figure 1 Unforgiving roadside environment [4]

The clear zone is a key safety concept used in road design, Figure 2. It represents the area that begins at the edge of each travelled lane and is available for emergency use by errant vehicles that run off the road. This zone includes any adjoining lane/s, road shoulders, verges and batters.
Generally, the width of the required clear zone increases as the design speed increases. On the basis of accident analysis in the Netherlands, the SWOV (Dutch Institute for Road Safety Research) has estimated that the minimum width of clear zones for three types of roads should be as follows [5]:
· 3.5 meters for single-lane regional highways;
· 7 meters for single-lane federal highways;
· 10 meters for motorways.

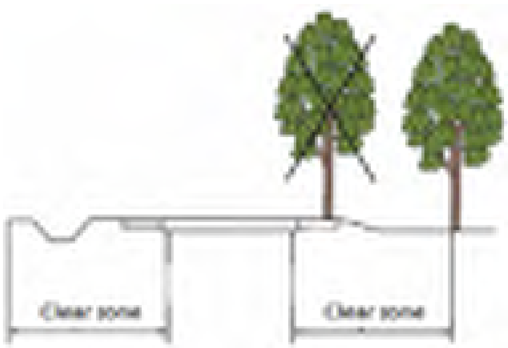

Figure 2 Roadside clear zone concept

The Clear Zone should be kept free of fixed, non-frangible hazards. It is not always possible, but all practical measures should be taken to provide this. Acceptable alternative options include safety barriers and physical measures to reduce travel speed. Less desirable alternatives include the use of narrower Clear Zones and compensatory measures such as delineation improvements.

2.2 Safe drainage structures

The drainage structures are an essential element of roads. They are designed to collect the pluvial water, but unfortunately they are very dangerous for road users. Because of the high water volume, they were designed very deep and with a high lateral slope of the walls, and in some cases they were even made of concrete, Figure 3.

Figure 3 Unsafe drainage structures

The development on new drainage systems which can cope with the expected amount of rainfall, yet don't create unsafe conditions for traffic users is not an easy task, but it is a necessary compromise. In developed countries, permeable drainage systems are commonly used in the proximity of roads, following the idea that they will be more dry than wet, even in areas with heavy rain. If the side of the trench is porous, evaporation is faster, and not just for the water seeped through the sides of the trench in the ground, but for any other type of infiltration. This is also a sensible ecological measure and it is called shallow green ditches. It has been discovered that most of the critical pollutions of the water from the road, such as oil and petrol will be destroyed by soil bacteria. For the case in which the water permeability of the soil is low, a subsurface piped solution is recommended.
Increasing the distance between the drain and the road will reduce the likelihood of a stray vehicle entering the drain and will provide room for pedestrians and other vulnerable road users away from motorised traffic, Figure 4. The ideal distance depends upon the usage of the road, but a typical recommended distance is 1.5 metres.

Figure 4 Safe drainage systems

2.3 Safe barrier systems

A barrier system is represented by a series of posts and cross beams, usually steel but sometimes concrete or wood, used to physically prevent vehicles passing a defined line, typically the edge of a road or the median line. Although larger vehicles may ride over barriers (Figure 5), they can be effective against the majority of motorized and non-motorized vehicles. However, the effectiveness of a barrier and indeed the danger that it may pose depend on how well it has been designed, located and installed.

Figure 5 Unsafe barrier systems

Safety barrier ends are usually considered hazardous when the termination is not properly anchored or ramped down in the ground or when it does not flare away from the carriageway. Crashes with "unprotected" safety barrier ends often "unforgiving" can result in a penetration of the passenger compartment with severe consequences.
Barrier systems should be used in areas where in case of a run off accident, the consequences of leaving the road by a vehicle would be much worse than if it would hit the parapet. The main role of the barrier systems is that they diminish the severity of a run-off-road accident when they are good designed and installed. For this to happen, they must have the property to absorb the shock of impact and prevent bouncing a vehicle back on the road just after a collision, as can be seen in the Figure 6.

Figure 6 Safe barrier systems with proper ends

The median barriers are designed to avoid front collision between vehicles travelling from opposite directions, but also they have an impact on pedestrians, as they encourage them to use safer areas to cross the road, Figure 7. A distinction needs to be made between the medians used to guide directional traffic management and those used for safety reasons. The second category must have a more solid construction, since their function is to divert vehicles tending to go over the median axis and absorb as much of the kinetic energy during the collision. To prevent the execution of u-turn maneuver on national roads with intense traffic, there have been installed on the median axis plastic barrier systems, in the colors red and white, filled with sand. Their presence on the roadway was properly marked with signs and road markings.

Figure 7 Safe median barriers

2.4 Rumble strips

Shoulder rumble strips have been proven to be a low cost and extremely effective treatment in reducing single vehicle run-off-road crashes and their severity, Figure 8.

Figure 8 Shoulder rumble strips

For rural freeways the Crash Modification Factor for the use of milled rumble strips has been estimated combining different studies in [1]:
- 0.89 (which means potential reduction of crashes of 11%) for single vehicle run-off-road crashes, with a standard error of 0.1;
- 0.84 (which means potential reduction of crashes of 16%) for single vehicle run-off-road fatal and injury crashes, with a standard error of 0.1.

For rural two lane roads the Crash Modification Factor for the use of milled rumble strips has been estimated combining different studies in [1]:
- 0.85 (which means potential reduction of crashes of 15%) for single vehicle run-off-road crashes, with a standard error of 0.1;
- 0.71 (which means potential reduction of crashes of 29%) for single vehicle run-off-road fatal and injury crashes, with a standard error of 0.1.

Given the very low standard errors these results can be considered extremely reliable in estimating the potential effect of milled shoulder rumble strips on these types of roads.

2.5 Frangible poles

Where it is not feasible to eliminate roadside hazards, it is possible to make them less injurious by changing their design as long as this takes account of real world accident data and current vehicle design. Frangible poles can be effective in reducing the severity of pole related crashes. These types of utility poles are specifically designed to collapse or break away on impact and reduce the severity of potential injuries. Two types of frangible lighting poles are most used are [6]:

1 Slip-base type poles;
2 Deformable poles.

2.5.1 Slip-base tire poles

Slip-base poles are widely used on freeways and other high speed roads in many countries and they are becoming more widely used. The slip-base poles consist of a normal pole stem, catering for mounting heights up to approximately 15 m. The base involves two plates clamped together with three equally spaced bolts. These plates and bolts are released during an impact, allowing the pole stem to break away from its foundation with minimal impact on the vehicle. The decision to use slip base poles will depend on the space available and the resultant likelihood that a falling pole would cause injury to other users of the roadside area. For example, a slip base pole will usually be inappropriate where pedestrian or cyclist traffic is common because a falling pole would pose an unacceptable risk to those road users, Figure 9.

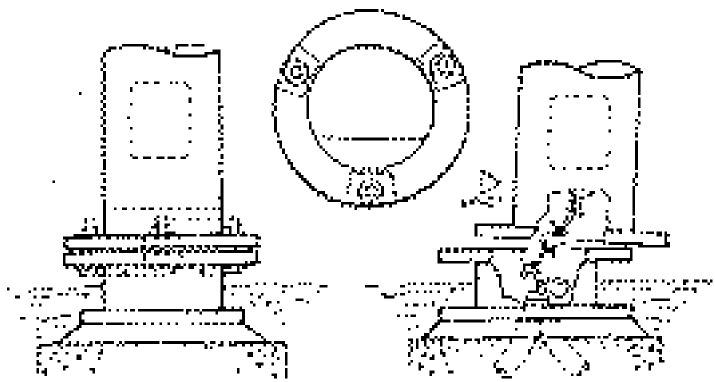

Figure 9 Slip-base poles [7]

2.5.2 Deformable poles

Deformable poles (Figure 10) provide a satisfactory degree of crash worthiness at lower vehicle impact speeds (ex: up to 80 km/h). They are particularly suited to low vehicle speed and/or high pedestrian activity areas. Impact absorbing poles differ from slip-base type poles in that in a vehicle impact they remain attached to the base structure and absorb any impact energy. The deformation of the pole is controlled by designed weakening of the pole stem over the lower 4m to 5m length.

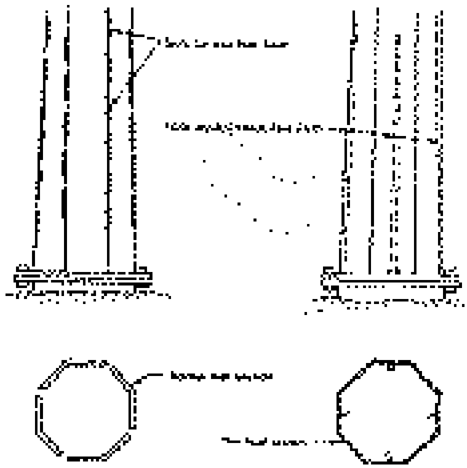

Figure 10 Deformable poles [7]

3 Conclusion

Forgiving road environments constitute a basic tool in preventing or mitigating an important percentage of road accidents related to driving errors. As everybody makes mistakes, drivers will eventually keep doing bad manoeuvres or actions. Over 80% of accidents are related to driver's error and about 25–30% of fatal accidents involve crashes with fixed roadside objects. This type of accidents is mainly caused due to driving errors that lead to lane/road departure. The existence of a forgiving road environment would prevent accidents of this type (and generally accidents that involve driving errors) and/or would reduce the seriousness of the consequences of such accidents. The first step in creating a forgiving road environment is the identification of error patterns that lead to accidents, in order to conclude to measures to be taken for rendering a road environment of forgiving nature. What is of outmost importance is to select the appropriate measure for each type of error, in terms of infrastructure enhancement. This paper has briefly reviewed concepts for designs of typical roadside elements. Although many nations have already begun to recognize and implement the "forgiving roads" concept, there are still perplexing questions regarding the nature and the extent of roadside treatments for a specific type of road and the cost effectiveness of this type of activity.

References

[1] ERA-NET ROAD: Safety at the Heart of Road Design, Final Report of the ERA-NET programme, May 2012.

[2] E. Bekiaris et al.: Infrastructure and Safety in a Collaborative World, 2011.

[3] European Transport Safety Council: Forgiving roadsides, 1998.

[4] Search Corporation and Universinj: Road Safety Manual for the Republic of Moldova, 2011

[5] SWOV: Roadside design in the Netherlands for enhancing safety, International Conference on Traffic Safety of two Continents, Lisbon, Portugal, 1997.

[6] NSW Centre for Road Safety: Reducing trauma as a result of crashes involving utility poles, August 2009.

[7] Department of infrastructure, Energy and Resources: Road Hazard Management Guide, Tasmania.

Recording and evaluation procedure of drivers' distraction in accordance with driver's characteristics in high speed arterials

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Abstract

Over the last years, distracted driving constitutes a considerably increasing road safety problem with disastrous results and it possesses a leading position among the accidents causes. The present study deals with driver's distraction due to out of the vehicle factors as well as factors related to the driver such as age, gender, driving experience etc. Considering exterior factors as the most significant, we can group them in four categories: built roadway, situational entities, the natural environment, and the built environment. Regarding the fourth category, it is related to the wide variety of civil infrastructure, the commercial land use combined with the high vehicle speeds. All these contribute to the setup of a very dangerous environment by increasing driver's distraction and inattention. This research is based on a medium scale experimental procedure which took place in three urban freeways in Greece, using a sample of 77 drivers. The distraction of the driver's attention is evaluated via a continuous recording of his gaze which acts as the main indicator regarding driver's performance. The main objective of this paper is to assess the side effects of roadside advertising and overloaded informational signs to driver's distraction and inattention. The results of this type of research procedures are very useful as a tool to prevent the forthcoming pressure for more and more billboards and trademarks on the roads as well as to encourage the adaptation of more precise regulations with regard to the road infrastructure, the placement of roadside elements, etc.

Keywords: Driving, distraction, advertising, billboards, naturalistic, research

1 Introduction

Distracted driving is a growing road safety problem and also the leading cause of car accidents. The nature and the specific characteristics of distraction have not yet totally clarified but the most of the route has already been covered as many researchers try to obtain all the necessary information to manage this phenomenon. The most crucial information, though, is about the nature of distraction and the dilemma whether or not distraction is differentiated from inattention. According to Regan et al. (2010) the taxonomy of inattention includes five major categories: Driver Restricted Attention (DRA), Driver Misprioritised Attention (DMPA), Driver Neglected Attention (DNA), Driver Cursory Attention (DCA), Driver Diverted Attention (DDA). Driver distraction as it is defined in this research is the last category of Regan's taxonomy.

2 Basic characteristics of distraction

The first step to a proper approach is to understand the basic characteristics of distraction as it appears in general. Many researchers have tried to define driver distraction and as a result the related literature contains a significant number of those definitions. In the first International

Conference on Distracted Driving (Hedlund et al. 2005:2) the scientific community agreed on a definition for distracted driving: “Distraction involves a diversion of attention from driving because the driver is temporarily focusing on an object, person, task, or event not related to driving, which reduces the driver’s awareness, decision-making, and/or performance, leading to an increased risk of corrective actions, near-crashes, or crashes”.

Distraction may be visual, cognitive, biomechanical and auditory (Ranney et al. 2001). Distraction at all forms, has become object of research recently, with distraction from a secondary task concentrating most of the research on the subject, particularly after the widespread use of mobile phones and the integration of driver assistance systems in modern vehicles. Naturally, priority is given to drivers of passenger cars without overlooking the other road users’ categories such as truck drivers, motorcyclists, bicyclists etc (Misokefalou et al. 2010).

The main causes of distraction are classified into two categories: Those coming from the interior of the vehicle and those from the external environment. In the second category, one finds some very important potential sources of driver distraction which can be grouped in four major categories: built roadway, situational entities, the natural environment, and the built environment (Horberry et al. 2009). According to Horberry et al. one component of built environment which consists the fourth category of the previous categorization is the billboards and any other kind of road advertising. In the case of causes related to advertising, it should be particularly emphasized that the purpose of their presence at some point at the roadside, or even in a moving vehicle in the road, is to capture the driver’s gaze in order for him/her to devote the required time so as to assimilate the information obtained. Roadside advertising billboards are designed by their very nature to attract attention. Crucially, though, the related potential threat to road safety is generally not acknowledged by the industry (Crundall et al. 2006). This is the reason why distraction by advertising is a very significant road safety phenomenon which must be elucidated by using all the available means and methods.

3 Frequency of driver distraction in crashes

The importance of this issue emerges from data which shows distraction from a secondary task as a cause of serious accidents as well as crashes. Particularly for billboards, Crundall in his study (Crundall et al. 2006) supports that though it is acknowledged that research into advertisement distraction has been extremely limited (Beijer et al. 2004), the few studies that have been conducted have demonstrated that drivers do look at and process roadside advertisements (Hughes et al. 1986), and that fixations upon advertisements can be made at short headways or in other unsafe circumstances (Smiley et al. 2004). Previous studies of accident statistics have also identified external distractors, including advertisements, as a significant self-reported cause of traffic accidents (Stutts et al. 2001). Particularly, for roadside distractors, evidence is mounting that roadside distractions (and advertising in particular) present a ‘small but significant’ risk to driving safety (Lay 2004). Conservative estimates collated from a review of several accident databases put external distractors responsible for up to 10% of all accidents (Wallace 2003). This is confirmed also by a recent simulator study (Young et al. 2009) in which there is a tentative suggestion that more crashes occur when billboards are present.

4 Method

4.1 Methods of evaluating driver distraction

The use of standardized methods gives the researchers the possibility to exchange data, conclusions and best practices. Therefore, it is important to detect the most suitable method of data collection (Rockwell. 1998). This aim can be achieved through a comparative study between the allocated methods, examining the advantages and disadvantages of every method separately as well as the usefulness and necessity of the results that every one of them produces as the only certain way for the researcher to detect driver's distraction is via the results that distraction produces. An analysis of this kind was made by the Univerity of the Thessaly (Eliou et al. 2009). During this study all the available methods were examined. The most popular among the available methods are based on elements of accidents, on experiments, on observation and surveys. Furthermore, there are some methods that are not included in any of the previous categories like Peripheral Detection Task and Visual Occlusion.

4.2 Selection of the appropriate method

The method considered the most appropriate is an observational-naturalistic study, which takes place in the field, using specially equipped vehicle to record the driver's eye movements and measure the frequency and the duration of the glances at every object considered potential source of visual distraction. The available equipment (Facelab machine) is capable of making continuous data recording. The main advantage is that with this method, in contrast with all the others categories, driving comes as close to the real thing as possible which is important for the research when we study human reactions. Naturally, there are some limitations both in designing and carrying out the experiment. The most important of these is the limited number of participants in comparison with other methods like questionnaires study, the unfamiliar vehicle which causes stress to the driver, the anxiety because of the sense of being monitored as the vehicle is equipped with cameras and, finally, the subjective discretion of the analyst-observer at the data processing.
Captiv software, which is compatible with FaceLab L2100, was used for the analysis of the results. This software gives the opportunity to analyse the data in detail by recording the number of glances at every billboard as well as the total time that the billboard captured driver's gaze during driving. At this point it should be noted that as distraction, in this study, is considered the continuous or intermittent but repeated capture of the gaze from a theme for longer than a total of two seconds as glances that last more than this time are related to driving errors (Rockwell, 1998). Smaller time intervals (0-0.7 seconds, 0.7-1.0 seconds, 1.0-1,6 seconds, 1,6-2.0 seconds >2 seconds) were also studied in order the results to be comparable with the results of other related studies. Highly experienced analysers carefully analysed the produced by Facelab machine videos and all the data were statistically analysed.

5 Experimental site

The research took place from June 2011 to February 20102, in 3 suburban roads in Greece. The first road is Attiki Odos, a freeway in the city of Athens. There were three routes under observation with a length of 19, 16.8 and 15 km respectively. The second road is Leoforos Kifisou , an arterial road in the city of Athens and the route under observation has a total length of 10 km. The third road is the national road between the city of Thessaloniki and the city of Giannitsa. The studied section is 7.7 km. In the last two roads, drivers drove both directions. The flow of vehicles is continual without being interrupted by traffic lights. The speed limit of the road is 90km/h. The total number of the points at which the data were collected is 136, of which the first 69 are at the first route, the following 40 at the second route and the last 27 at the third route. In this study other objects besides the advertisement has been selected to participate in order to act as comparison points to the advertising.

6 Participants

Using volunteer drivers, who are required to drive a car on the three selected roads, under the supervision of the researcher, who was always in the passenger seat checking the proper function of the system, the obtained results are characterized by a high degree of reliability and validity. Seventy seven drivers participated in the surveyof which 62% are men and 38% women.

The drivers were selected with criterion their familiarity with the selected roads in order to eliminate the stress which an unfamiliar route induces. All drivers were familiar with the road, as they use it in a daily basis, but the subject of the study was completely unknown to them. Regarding the limitation of the unfamiliar vehicle, each one of them, in order to become familiar the vehicle drove the selected route 2 times before the third run which was the one that we focused our attention at during the analysis process. The drivers are between 26 and 45 years old and the vast majority of them, in all three traffic routes, possesses a driving license and drives systematically for more than ten years.

7 Material - Data collection

The equipment used in the survey was very carefully chosen in order to produce the optimal quality, completeness and integrity of results. It includes a passenger vehicle and a monitoring and recording system (Facelab, Seeing machines), which detects and records every single movement of the driver's gaze and the driver's head. It is composed of two cameras for the recording of the above and an external camera for the recording of the road scene. All measurements for the experiment took place during the day, under normal traffic conditions as well as normal weather and lighting conditions.

8 Results

In this study, the information isolated and analysed in depth, is related to the external impulses that cause driver distraction and concentrates interest mainly on billboards near the road and their role in driver's distraction of attention. For this purpose, many sites of interest related to advertising along the road were identified and mapped for routes of every road. Among them there are billboards, banners, soundpannels with graffiti and posters on them and gas station signs. Additionally, other elements on the road, such as road signs and buildings were parts of the study. The analysis included an examination of driver behaviour, as far as concern the reactions of drivers' pupils of the eyes while driving under the existence of these potentially evocative distraction elements of the road environment.
The following 3 figures (Fig. 1,2 and 3) show the results from the analysis of the gaze direction to the observed points of each route. Each driver drove the selected route 3 times but we decided to focus our attention at the third one because of the familiarization of the driver with the vehicle which was analysed at the method section. As it shows, there are certain points on the route which attract attention more than the allowed, from the aspect of safety, time.
The detailed analysis of the data came from the eye gaze, in terms of glance duration, led us to the conclusion that the selected points could, in their majority, characterized as attractors with an intense role in the driver visual field. Furthermore, from the four major categories under research, the one that seems to capture driver's eye the most is billboards. Furthermore, there are many of them that attract the gaze for more than 1,6 seconds, a time interval that according to Lee (2007) is dangerous for the safe implementation of the driving task. The time frame of the 1 second that many researcher like Zwalen (1988), Rockwell (1998) and Wickman (1998) characterize as the safe limit, is being surpassed at most of the points. If we decide to adapt Beijer's (2004) opinion about safe eye behaviour, according to which glances for more than 0,7 seconds away from the driving task could be unsafe for the execution of it, all points with minor exceptions could be considered as possible risk points.
The multiple regression examined if the 24 variables under research (age, gender, driving experience, number of point, category, number of lines at the point, direction of the point, object into a tunnel, position of segmentation, distance from the road, luminosity, number of objects at the point, median barrier, position, size, emergency lane, road category, time, weather conditions, distance from the start, distance from the leading vehicle, traffic volume, speed, number of glances) can predict drivers' total distraction time.

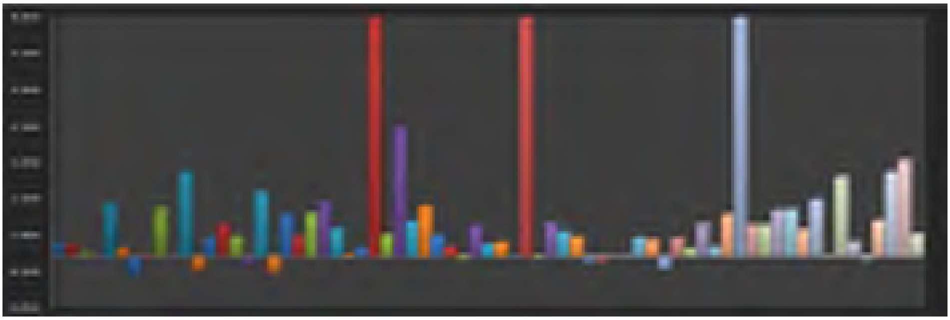

Figure 1 Mean of the time interval dedicated to each point at Attiki Odos road

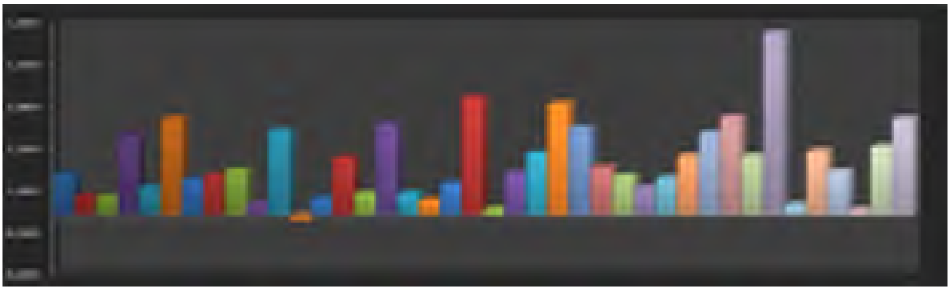

Figure 2 Mean of the time interval dedicated to each point at Leoforos Kifisou road

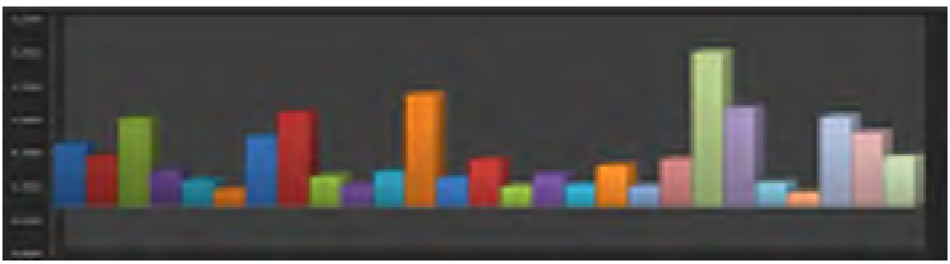

Figure 3 of the time interval dedicated to each point at Thessaloniki – Giannitsa road

The multiple regression produced a weak fit with the data ($R^2 = 0.038$ & adjusted $R^2 = 0.030$) most probably due to the number of variables in the model and thus in the equation. Still, the combined influence of the variables was an excellent predictor of the total time producing a statistically significant result ($F(24, 2840) = 4.658$, $p<0.01$). Specifically, the statistically significant variables are the following:

Significantly positive relationships were identified between the number of the points: $t(2840) = 3.736$ with $p<0.01$ (as the number of the point increases by one unit, the total distraction time will increase by 0.007 seconds), the size of the object: $t(2840) = 2.111$ with $p=0.035<\alpha=0.05$ (as the size of the point increases by one unit, the total distraction time will increase by 0.153 seconds) and the number of the glances: $t(2840) = 4.450$ with $p<0.01$ (as the number of glances increase by one unit, the total distraction time will increase by 0.15 seconds).

Significantly negative relationships were identified between the distance between the point and the road: $t(2840) = -3.718$ with $p<0.01$ (as the distance from the road decreases by one unit, the total distraction time will increase by 0.319 seconds).

Thus, the regression equation for the statistically significant results is the following:

$$\begin{aligned} y &= a+b\times 1+b\times 2+b\times 3+b\times 4 \\ &= 0.085+0.007\ \times \text{ no of point}-0.319\times\text{distance}+ \\ &\quad +0.153\times\text{size}+0.15\times\text{no of glances} \end{aligned} \tag{1}$$

The variables that gave statistically significant results at the regression are being further examined in order to conclude about their influence on the total distraction time.

9 Conclusion

Distraction of driver's attention during driving is a major road safety problem, which threatens not only the driver's safety but also the safety of other drivers and road users. The focus of the research on drivers of passenger vehicles is due to the fact that those drivers consist the largest category of road users with growing involvement in accidents, which are caused by the distraction of driver's attention. The goal of the research is to identify and clarify the causes, the frequency of appearance and the way that certain factors influence the distraction of attention of each driver, focusing on the role played by roadside advertising in Greece as a parameter of the distraction of the driver's attention.

The methods commonly used in a study of driver distraction aren't all feasible or effective to the same extent. The chosen method allows the continuous data recording with its main advantage being the fact that driving is as close to the real thing as possible. Thus, the results are characterized by a high degree of reliability and validity. It, also, gives the opportunity to the participant to have an adjustment period with the vehicle in order to obtain a normal driving behaviour. The small possibility of the researcher to control the situations and create desirable driving scenarios is among the disadvantages of this method. The environmental conditions, also, cannot be controlled. Another disadvantage is the increased cost of the method due to the eye tracker. Finally, as disadvantage of the eye tracker we could mention the difficulty of the installation in the car as well as its sensitivity to changes (e.g. lightness conditions).

This research concluded that all roadside billboards of the route and many other objects which belong to the categories "road signs" and "built environment" distract driver's attention in a degree which could be considered as dangerous. Among the road characteristics, the driver's characteristics, the characteristics of the point and the conditions of the measurement there are some variables that are statistically significant regarding the distraction of the driver's attention. More specific, the statistically significant variables are the number of the point, the size of the object, the number of glances at the point and the distance between the point and the road.

Much of the data analysis requires collaboration with experts such as psychologists and doctors in order to provide an integrated approach. Furthermore, a comprehensive policy to reduce the visual pollution near the roads, such as billboards, can help not only to improve the road aesthetic but also to significantly improve road safety by eliminating driver's visual distraction of attention.

The results of this type of research procedures are very useful as a tool to prevent the forthcoming pressure for more and more billboards and trademarks on the roads as well as to encourage the adaptation of more precise regulations with regard to the road infrastructure, the placement of roadside elements, etc.

To sum up, it is a fact that driver distraction is a major cause of accidents; therefore, the responsibility over the issue translates into efforts to reduce the number of injured and dead drivers. In order to achieve that, the solution of the distraction phenomenon could be found in the combination: "research-education-design- legislation".

This research is under further analysis, as a number of 24 variables - from the road, the conditions, the points and the driver - is available to the researchers and is being further statistically analysed in order to conclude about the contribution of the selected variables in driver distraction.

References

[1] Beijer, D.D., Smiley, A. and Eizenman, M.: Observed driver glance behavior at roadside advertising signs, Transport Research Record, 1899: 96–103, 2004.

[2] Crundall, D., Van Loon, E. and Underwood, G.: Attraction and distraction of attention with roadside advertisements. Accident Analysis & Prevention, 38: 671–677, 2006.

[3] Eliou, N. and Misokefalou, E. Comparative analysis of drivers' distraction assessment methods. Presented at 22nd ICTCT Workshop, Towards and Beyond the 2010 Road Safety Targets-Identifying the Stubborn Issues and their Solutions. Leeds, UK, October 2009.

[4] Greek Police.: Accident Statistics for the Year 2011, Ministry of Citizen Protection, Greece, 2011.

[5] Hedlund, J., Simpson, H.M. & Mayhew, D.R.: International conference on distracted driving-summary of proceedings and recommendations. International Conference on Distracted Driving, Toronto, Canada, October 2005.

[6] Horberry, T., Regan, M.A. and Edquist, J.: Driver distraction from roadside advertising: The clash of road safety evidence, highway authority guidelines and commercial advertising pressure. Presented at first International Conference on Driver Distraction and Inattention. Gothenburg, Sweden. September, 2009.

[7] Klauer, S.G., Dingus, T.A., Neale, V.L., Sudweeks, J.D and Ramsey, D.J.: The impact of driver inattention on near-crash/crash risk: an analysis using the 100-car naturalistic driving study data. Report No. DOT HS 810 594, National Highway Traffic Safety Administration, Washington, D.C, 2006.

[8] Lay, M.G.: Design of traffic signs, in The human factors of transport signs, edited by C. Castro and T. Horberry: Boca Raton, FL: CRC Press: 25–48, 2004.

[9] Misokefalou, E., Eliou, N. and Galanis, A.: Driving distraction of attention audit of motorcycle and bicycle users in urban areas, Case study in the city of Volos, Greece. Presented at the 5th international congress on transportation research, Volos, Greece, September 2010.

[10] National Highway Traffic Safety Administration.: Distracted driving 2009. Traffic safety facts, DOT HS 811 379. NHTSA's National Center for Statistics and Analysis, Washington, DC, 2010.

[11] Ranney, T.A., Garrott, W.R. and Goodman, M.J.: National Highway Traffic Safety Administration driver distraction research: past, present and future. Presented at the 17th International Technical Conference on Enhanced Safety of Vehicles, Amsterdam, The Netherlands, June 2001.

[12] Regan, A.M., Hallet, C., Gordon, P.C.: Driver distraction and driver inattention: Definition, relationship and taxonomy. Accident Analysis and Prevention, 43: 1771-1781, 2010.

[13] Rockwell, T.H.: Spare visual capacity in driving- revisited, in Vision in Vehicles II edited by A.G. Gale, H.M. Freeman, C.M. Haslegrave, P. Smith and S.P. Taylor. Elsevier : 317-324, 1998.

[14] Smiley, A., Smahel, T. and Eizenman, M.: Impact of video advertising on driver fixation patterns. Transport Research Record, 1899: 76–83, 2004.

[15] Stutts, J.C., Reinfurt, D.W., Staplin, L.W. and Rodgman, E.A.: The Role of Driver Distraction in Traffic Crashes. AAA Foundation for Traffic Safety, Washington, DC, 2001.

[16] Tasca, L.: Driver distraction: towards a working definition. Presented at International Conference on Distracted Driving, Toronto, Canada, October 2005.

[17] Wallace, B.: Driver distraction by advertising: Genuine risk or urban myth?. Municipal Engineer, 156:185–190, 2003.

[18] Wikman, A.-S., Nieminen, T., & Summala, H.: Driving experience and time-sharing during in-car tasks on roads of different width. Ergonomics, 41: 358-372, 1998.

[19] Young, K., Mahfoud, J.M., Stanton, N. A., Salmon, P.M., Jenkins, D.P. and Walker, G.H.: Conflicts of interest: The implications of roadside advertising for driver attention while driving: Skill and awareness during inspection of the scene. Transportation Research Part F 12: 381–388, 2009.

[20] Zwahlen, H.T., Adams, C.C. Jr., & Debald, D.P.: Safety aspects of CRT touch panel controls in automobiles, in Vision in Vehicles II edited by A.G. Gale, H.M. Freeman, C.M. Haslegrave, P. Smith and S.P. Taylor. Elsevier : 335-344, 1988.

AN APPROACH TO ASSESSING DRIVER'S BEHAVIOUR AT ROUNDABOUTS

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Abstract

In urban traffic, roundabouts have been considered as an alternative traffic facility that can improve safety and operational efficiency, compared to un-signalized intersection. But in Morocco where roundabouts are relatively new, the concept is not rooted in the road culture of drivers. Moroccan drivers are used to the traffic lights telling them what to do. Roundabouts involve drivers using their own judgments at an intersection, so roundabouts negotiation can be confusing for drivers who are not familiar with their use. This fact has guided our research in this area to an approach that takes into account the Moroccan road context and human factors in this context. This article presents a detailed analysis of driving task to determine key functions performed by drivers as they approach and navigate through roundabouts. Our analysis aims to understand the mechanism of driving performance in roundabout's environments, to predict how drivers might respond , to identify roundabout elements that play a role in incorrect roundabout negotiation and therefore to suggest measures for improving drivers' abilities to properly negotiate roundabouts.

Keywords: roundabout, human factor, cognition, task analysis, human functional failure

1 Introduction

Moroccan experience with roundabouts is rather limited to date, but their numbers are increasing rapidly. For further discussion some consensus about terminology seems to be useful in order to avoid mixing things. A roundabout is a form of intersection design and control which accommodates traffic flow in one direction around a central island, operates with yield control at the entry points, and gives priority to vehicles within the roundabout, [1]. In Morocco, the new highway code (2010) defines the roundabout as "an intersection where all traffic merges into and emerges from a one-way road around a central island (impossible to cross) of circular shape, the circulation on this roadway is in a counterclockwise direction". The geometric configuration of roundabouts, as compared to others intersections, promotes the traffic safety. In fact numerous studies conducted in Australia and Canada [2-5], where roundabouts are common, and in the US, have found that roundabouts provide several benefits compared to standard intersections. A study was conducted in US [6], in 2003, on the safety performance of 23 roundabouts, the authors reported highly significant reductions of 40 percent for all crash severities combined and 80 percent for all injury crashes. In addition to traffic safety, other advantages of a roundabout have been studied extensively in the following areas: Capacity and quality of traffic flow [7], Traffic- calming effects, reduced delay and concomitant emissions. [8-10] Although roundabouts provide many benefits, there are some disadvantages as well. The primary drawback is that roundabouts have design elements that go against the common rule-of-the-road expectancy to yield to vehicles on the right, which can lead to confusion and error for unfamiliar drivers. In order to properly address these questions, this article has been divided into four parts. After this introduction, the second section presents a detailed analysis of driving task to determine key functions performed by drivers as they approach and navigate through roundabout. Then from a literature review, we propose our driver error taxonomy at roundabout. We present in the third part our research approach and the primary findings of our study. The conclusion insists on the interest of our approach to understand better the difficulties that drivers encounter at roundabouts and how to address these difficulties in the context of improving roundabout safety.

2 Human factors issues in roundabouts

Negotiating roundabouts is one of the most complex and demanding tasks a driver faces. Even though roundabouts comprise just a small amount of the roadway surface area, they generally are more complex and difficult to navigate than most other road segments. This fact has guided our research in this area to an approach that takes into account the Moroccan road context and behaviour of the driver in this context.

2.1 Road user tasks and information requirements

Concerns for drivers at roundabouts include: confidence approaching the roundabout, navigating around the roundabout, direction of travel and yielding rules [11]. As driving involves complex interactions between human factors and system responses, the driver, through his senses can only selects what is significant for him, depending on his knowledge, on the situation and his objectives. While driving, many subtasks have to be performed at the same time, in an environment with many rules and complex interactions. Therefore, in order to determine exactly what a driver must do to consistently negotiate roundabouts safely a task analysis needs to be carried out. The technique of Hierarchical Task Analysis (HT A) is applied for collecting information on the tasks and procedures and for studying the working environment, the process of HTA is to decompose tasks into subtasks to any desired level of detail. Each subtask, or operation, is specified by a goal, the input conditions under which the goal is activated, the actions required to attain the goal, and the feedback indicating goal attainment, [12]. From an ergonomic perspective, the driver's behaviour in a particular situation is regarded as a function of information available at given moment (both information present in the road environment and information stored in the driver's memory); of its processing and of the decision-making criteria underlying the regulating action he takes,[13]. In the field of cognitive psychology, the notion of situation awareness (SA) can be formally defined as the "perception of the elements within a volume of time and space (Level 1), the comprehension of their meaning (Level 2), and the projection of their status in the near future (Level 3)", [14]. So each task encountered by the driver at roundabout involves a sequence of:(i) Perception or recognition; (ii) Decision making ;(iii) Execution or performance; and (iv) Real time system response by the vehicle, roadway and surrounding environment

2.2 Roundabout elements of concern to drivers

Roundabouts can be visually complex, requiring that drivers scan several different areas and keep track of several different elements to get the information they need to safely pass [15]. As a start point our research has been focused on the effects that traffic signs have on the driver performance and how to avoid distractions and human errors provoked by them.The objective of the road signs is to transmit an unambiguous message to the driver quickly and clearly, to minimize disturbance with the other users and to allow a sufficient time after recognizing the sign to make decision and control action. Drivers' perception processes are very important in understanding the effectiveness of a road sign. The principles that enhance perception and reaction to signs are: conspicuity, visibility, maintainability, legibility , and standardization, [16]. According to this perspective, Andreassen [17] found that "...if any link existed between the sign and an accident, it might be due to the poor design, maintenance or placement of the sign ...". On the other hand, a study [18] conducted in Morocco in urban areas has shown that road signs led to several problems, It has been noticed in this study that " urban environment suffers from multiple dysfunctions, roundabouts are designed without rigorous standards, also roundabouts with the same features have different roads signs from one city to another and even within the same city". This finding justifies that research should be conducted to further knowledge about the influence of traffic signs on car drivers' behaviour, and it is one of the main objectives of our study.

2.3 Roundabout driving task analysis: a driver operating model

This article presents a detailed analysis of driving task to determine key functions performed by drivers as they approach and navigate through roundabout. The technique of hierarchical task analysis described above has been used to analyse the task of negotiating a roundabout. Our analysis underlies the assumption that to properly drive through a roundabout, drivers need competences more diversified than only the respect of the rules of Highway Code. Since this is a change from standard intersections, many drivers have difficulties in their first few encounters with roundabouts. In our frame we have combined the task analysis approach with the concept of situation awareness. Within each segment (approach, entry, within the roundabout, exit), we have identified individual tasks that drivers should or must perform to safely navigate the roundabout. Each task encountered by the driver involves information that needed to be obtained, decisions that needed to be made, or actions that needed to be taken. The steps, a driver must consider to correctly manoeuvres through a roundabout include the following

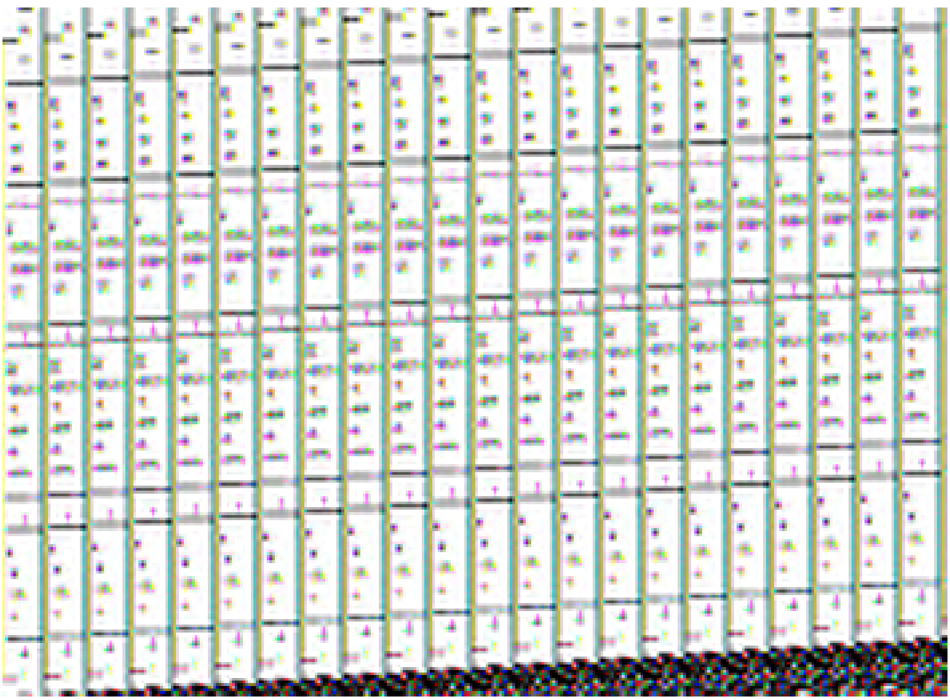

Figure 1 Frame of functional subtasks involved in roundabout negotiation.

This frame was used to construct the observation sheet of human functional failures for each of the task at roundabout, consistent with ergonomics concepts and specifically adapted to the roundabout driving task. Indeed, once correct driving behaviour at roundabout has been established, deviations from that behaviour can be defined as errors. Where an error occurs in any one or more of these steps, it may lead to an incident (such as a near miss) or accident (crash).

2.4 Literature review of driver error

Human error is a problem of great concern within complex system. One of the obvious consequences of assessing human error in roundabout is that, in understanding how and why it happened we may be able to prevent similar events. According to the literature [19], there have been numerous attempts at defining the construct of human error but no universally accepted definition exists. In order to avoid any semantic confusion, 'human error' will be considered in this article under the label of 'Human Functional Failure' (HFF), defined by Reason [20] such as following:"a generic term to encompass all those occasions in which a planned sequence of mental or physical activities fail to achieve its intended outcome". There are also a number of different error classification schemes and error taxonomies avaible. A more recent study conducted by the American driver and traffic safety Education Association [21], describes driver errors in relation to the three driver tasks of perception, decision and execution: (i) Perception or problem recognition errors included: driver failed to stop for sign, delays in problem recognition (e.g. Improper lookout, internal distraction, delays in recognition, inattention, external distraction); (ii) Decision error included: Excessive speed, false assumption, improper technique /practice, improper maneuver, inadequate signal, tailgating, misjudgment of distance /closure, failure to turn, excessive acceleration ; (iii) Execution or performance errors that refer to : Improper evasive action, inadequate directional control, overcompensating, panic or freezing, critical non-performance(e.g. passing out, falling asleep). According to this study the types of drivers errors cited above are also influenced by a range of personal, environmental or infrastructure factors, which may contribute to drivers' error. These factors include: (Inadequate knowledge, skills, and training; Impairment due to: dysfunctions, disabilities; Willful inappropriate behaviour; Infrastructure, environment problems)

2.5 Proposed Human error taxonomy when crossing roundabouts

HAZOP (Hazard and Operability Study) is a classic tool of the industrial world that was developed for use in process design audit and engineering risk assessment. HAZOP involves analysts applying guidewords, such as not done, more done or later done, to each task step in order to identify potential errors that may occur.
In our study, a set of human error HAZOP guidewords were used, each guideword is applied to each subtask at roundabout. Once a description of error is provided, a cause and types of the error are described based on an orview of the literature which were interested to the sources of driver errors at roundabouts [21-28]. The driver error taxonomy is presented in table 1.

Table 1 The driver error taxonomy

Nº	task	Domain	Internal error mode	Deviation	Psychological error mechanism	Errors types
1	**Subtasks on approach**					
1.1	Perceive roundabout cues	Perception	Miss see No detection (visual) See too late	User fails to perceive roundabout	Expectation Vigilance Distraction Overload Impairment due to dysfunctions	Recognition errors
1.2	Recognise presence of roundabout;	Memory	Forget information Miss recall information Wrong information obtained	User fails to recognise roundabout	Memory confusion Overload Misinterpretation Knowledge problem (user is unfamiliar with roundabout)	Recognition errors
1.3	Position correctly and adjust speed	Decision Action	Action omitted Action too late Action too early	Failing to formulate safe stopping strategy Failing to control speed on approach	Distraction Inadequate model mental Knowledge problem (user is unfamiliar with roundabout) Impairment due to dysfunctions	Decision errors And performance errors
2	**Subtasks on entry**					
2.1	Be aware of other vehicles and objects in the environment (visual checks)	perception	Miss see No detection (visual)	Failing to stop behind a queued vehicle	Expectation Vigilance Distraction Time pressure Impairment due to dysfunctions	Recognition errors
2.2	Yield the right of way to traffic already in the roundabout	Action	Action omitted Action too late Deliberate violation	Failing to yield the right of way to vehicles in the roundabout	Confusion Distraction Inadequate model mental Knowledge problem (user is unfamiliar with roundabout) Impairment due to dysfunctions	Decision errors
2.3	Assess adequacy of gaps for entering	Decision	misprojection poor strategy late strategy	Accepting an unsafe gap distance	Misinterpretation Knowledge problem Decision overload	Decision errors
3	**Subtasks Within the roundabout**					
3.1	Maintain proper lane position	Decision Action	Action omitted Action too late Wrong action	Failing to detect proper lane assign-ment when entering	Inadequate model mental Knowledge problem (user is unfamiliar with roundabout) Impairment due to dysfunctions	Decisions errors and Performance error

Table 1 The driver error taxonomy (continued)

3.2	Be aware of other users acceleration/ deceleration	perception	Miss see No detection (visual)	Failing to avoid conflicts with other users	Expectation Vigilance Distraction Time pressure Impairment due to dysfunctions	Recognition errors
3.3	Travel counter-clockwise;	Action	No performed action Deliberate violation	travelling clockwise	Confusion Time pressure Knowledge problem	Performance errors
3.4	Keep moving and make a stop/go decision in the dilemma zone	Action Decision	decision omitted decision too late Wrong decision	Stop at roundabout and refusal of priority	Confusion Distraction Inadequate model mental Knowledge problem (user is unfamiliar with roundabout) Impairment due to dysfunctions	Performance errors and Decision errors
4	**Subtasks on exit**					
4.1	Maintain lane position	Action	Action omitted Action too late Deliberate violation	Changing lanes incorrectly when exiting	Confusion Time pressure Knowledge problem Decision overload	Performance error
4.2	Use right turn signal to signal intent to exit	action	Action omitted Action too late Wrong action	Signal driving direction omitted Failing to use right turn signal to signal intent to exit	Distraction Knowledge problem (user is unfamiliar with roundabout)	Decision error
4.3	Exit at a slow speed.	Action	Action omitted Action too late Wrong action	Failing to control speed on exit	Confusion Time pressure Knowledge problem	Performance error

3 Methodology and summary finding

3.1 Study methodology

Our study was conducted at three roundabouts in Rabat. The three roundabouts met the primary characteristics of modern roundabouts: They had no traffic-control signals other than yield signs for entering traffic; circulating vehicles had the right of way. An observation sheet was developped for the field study. For each task involved in crossing roundabout, the observer had to chek if all subtasks required were accomplished by the subject and to mark missing, incorrectly or unrequired one. The observer also counted the frequency of different manoeuvres of the drivers, (e.g. lane-changing, overtaking). These measures served as basic exposure information so that error rates for different driving tasks could be calculated. In addition to the observation method described above, a questionnaire has been used to gather information about the reliability of drivers in the driving task under investigation. The following results will not draw upon the whole set of methods, but will mainly rely on the driver observation method.

3.2 Summary of finding

The roundabouts inspections that were carried out within the scope of this research had a rather exploratory character. The findings of the study can be summarized as follow in chapters 3.2.1 and 3.2.2.

3.2.1 Roads Signs

From a human factor perspective, some aspects of the current signage at roundabouts visited are not optimal, indeed:

- Roads Signs are not always conspicuous due to their size, colour and position;
- Warning sign which indicates the presence of a roundabout ahead wasn't always used at roundabouts visited;
- At some locations a mixture of road signs and other signs for the public could be seen as confusing or conflicting. An exemple is an "advertising sign"posted with a "yield sign"at one of the roundabouts visited;
- It was noted that "yield signs"were worded differently on each roundabout. The sign on one roundabout reads "give way", whereas the sign on another roundabout reads "you don't have priority". This discrepancy increases the ambiguity of the instruction.

3.2.2 Drivers' behaviour

The sample established was composed by 136 vehicles observed. Drivers' behaviour shown below represents the range of driver failure experienced at the roundabouts visited. These failures do not represent every possible error but represent easily identifiable deviation that can be related to incorrect roundabout negociation as previously defined. The analysis of the data collected shows that:

- Almost 88% of the drivers observed were judged to be at fault by failing to signal intent to exit;
- 79% of drivers omit signal driving direction;
- 72% of drivers fail to yield the right of way to vehicles in the roundabout;
- 42% of drivers fail to detect proper lane assignment while 34% change lane incorrectly;
- 54% of drivers fail to control speed on approaching roundabout.

Additionally, the results indicated that the least frequently reported aberrant behaviours at roundabouts visited were:

- Stopping at roundabouts and refusal of priority among 15% of drivers;
- Travelling clockwise among 5% of drivers.

4 Conclusion

The roundabout, with all kinds of traffic users, is too complex and it is extremely difficult to define a solid limit between correct and incorrect behaviour, hence the need to set up developments which allow the driver to discern, to identify and to choose easily, in this environment, the indices for the effective regulation of its activity. The methodology we have presented here represents an analytical approach. The interest of this approach is that it attempts to obtain an overview of drivers' behaviour in specific driving situation (e.g. roundabouts). The conclusions of the preliminary findings of our study indicate that the most common deviations occurring at roundabouts in this analysis were: lack of knowledge of priority rule and omission of signal driving direction. These results address that there is a general lack of awareness and understanding on the part of most drivers regarding roundabout. This is enhanced by inconsistencies in the road signs at roundabouts, which can be confusing and misleading. More generally it can be emphasized that roads signs at roundabouts need to be uniform from roundabout to another. This gives drivers the opportunity to gain experience

with roundabouts and at the same time develop mental schemata for managing roundabouts that help them safely negociate roundabouts, [29].
It is clear that there is much further investigation is required on the causal factors of errors and on the implications that these driver errors have on roundabout safety. The second stage is now to address the relationship between the elements of the roundabout environment and types of driver errors, this work is on progress by using questionnaires to make drivers precisely explain their perceptions of the facts, their decisions, actions and the difficulties they encountered, with the aim to suggest measures for minimising potential driver's errors and improving drivers' abilities to properly negotiate roundabouts in Morocco.

References

[1] Thaweesak, T.: Modern Roundabouts for Oregon, Technical Report Form DOT F 1700.7, June 1998.

[2] Beaupré, A.: Evaluation et recommandations pour le développement des carrefours à sens giratoire au Québec, Montréal, 2011.

[3] Schermers,G., Wegman, F., Vliet,P., Horst,R. & Boender,J.: Country report, the Nederlands, 4th international symposium on highway geometric design,Valencia, june,2010.

[4] Adams, P.: Traffic signals and roundabouts: Are they really safer? Roads & Transport Research, ARRB, Victoria, December 1995.

[5] Jorgensen, E. & Jorgensen, N.O.: Traffic Safety at Danish Roundabouts Constructed after 1985, VTI Konferens, n° 4A, part5. Sweden, 1996.

[6] Persaud, B., Retting, R., Garder,P. & Lord,D.: Observational before-after study of the safety effect of U.S. roundabout conversions using the empirical bayes method, 2003.

[7] Rahman, M. & Hicks, T.: A critical look at roundabouts, Compendium of Technical Papers, ITE, 1994.

[8] Elvik, R., Mysen, T.: Traffic safety Handbook (Norwegian). Institute of transport economics. Norway, 1997.

[9] Garder, P.: The Modern Roundabout, the Sensible Alternative for Maine Department of Civil and Environmental Engineering, University of Maine, Orono, 1997.

[10] Retting, R. A., Luttrell, G. & Russell,E.: Public Opinion and Traffic Flow Impacts of Newly Installed Modern Roundabouts in the United States, In TRB 2002 Annual Meeting CD-ROM. CD-ROM. Transportation Research Board, Washing, D.C., 2002.

[11] Devlin, A., Candappa, N., Corben, B. & Logan, D.: Designing safer roads to accomodate Driver error. Curtin-MONASH ACCIDENT RESEARCH CENTRE,2011.

[12] Fastenmeier, W. & Gstalter, H.: Driving task analysis as a tool in traffic safety research and practice, safety science, volume 45, issue 9, November 2007.

[13] Annett, J.: Hierarchical Task Analysis, the handbook of task analysis for human-computer interaction, edited by Diane Diaper and Neville Stanton.

[14] Endsley, M. & Kaber, D.: Team Situation Awareness for Process Control Safety and Performance, Process Safety Progress, Vol. 17, No. 1, 1998.

[15] Le Thu Huyen, M.: Risk analysis, driver behaviour and traffic safety at intersections in Motocycle-Dominated traffic flow. Tuprints.ulb.tu-darmstadt, 2009.

[16] Thomson, R.: Ranking of recommendations for road infrastructure safety. TREN-04-FP6TR-S07.36996. Spain, 2008.

[17] Andreassen, D.C.: Traffic accidents and advertising signs. Technical Note No. 1, Australian Road Research, 15(2), 103-105,1985.

[18] Boulaajoul, B.: Sécurité routière au Maroc : Bilan et perspectives, Rapport de la commission économique de l'Afrique, juillet 2009.

[19] Salmon, P., Regan, M. & Johnston, I.: Human error and road transport: phase one- literature review, Report n° 256, December 2005.

[20] Reason, J.: Human error, Cambridge University Press, New York, 1990.

[21] Faucett, J.: Remedies for driver error, Final report 567, Arizona, August 2004.

[22] Summala, H., Pasanen, E., Räsänen, M., & Sievänen, J.: Bicycle accidents and drivers' visual search at left and right turns, Accident Analysis and Prevention, 28(2), 147-153, 1996.

[23] Räsänen, M., & Summala, H.: Car drivers' adjustments to cyclists at roundabouts. Transportation Human Factors, 2(1), 1-17, 2000.

[24] Nachtergaêle, C. & Fleury, D.: Analyse approfondie de l'accidentologie d'aménagements urbains, Rapport de convention INREST/RE-03-909-FR, juin 2003

[25] Isaac, A., Shorrock, T., Kennedy, R., Kirwan, B., Andersen, H. & Bove, T.: Technical Review of Human Performance Models and Taxonomies of Human Error in ATM (HERA), EATMP Infocentre, Brussels, 2002

[26] Rasmussen, J., Duncan, K. & Leplat, J.: New technology and human error. London. 1987.

[27] Understanding human factors and developing risk reduction solutions for pedestrian crossings at railway stations, report T730, www.rssb.co.uk,2009.

[28] Research into signs and signals at levels crossings appendices A to F, Rail safety and standard board LTD, www.rssb.co.uk,2011.

[29] Davidse, R.: Assisting the older driver: Intersection design and in-car devices to improve the safety of the older driver, doctoral thesis, SWOV Institute of road safety research, 2007.

HOMOGENIZATION OF SPEED ON SECONDARY AND LOCAL ROADS IN THE FLANDERS REGION: AN EXPLORATORY STUDY MAKING USE OF A TRAFFIC SIGNS DATABASE

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Abstract

Speed is an important factor of road accidents. This paper focusses on changes and differences in speed of the traffic flow and on speed differences between different vehicles. Based on an international literature survey the relevance of the speed related factors for road safety are described. Road categorization and delineation of speed limit zones are identified as the most important tools to obtain a more homogeneous speed pattern. Based on a series of case studies including the different urbanization typologies, it is shown that regarding the implementation of these two planning concepts, little has been achieved in Flanders. The scattered and ribbon shaped urbanization along the secondary and local roads and the step by step adaptation of speed limits on road segments by the different road authorities, give rise to a manifest lack of homogeneity . Based on the Flemish Road Signs Database a quantitative analysis (e.g. relating to the length of segments, number of speed limit changes on a road section, etc.) could be carried out. The database contains all traffic signs relevant for speed limits on the entire road network in Flanders, including the road sign position and the description of its characteristics. Based on the case studies both an analysis of the existing situation as well as an explorative assessment of a generic scenario (entering a general speed limit of 70 km/h in non-built up areas instead of the current limit of 90 km /h were described. By implementing this scenario, six out of seven speed limit signs can be removed and the workload for the drivers can be reduced proportionately. So speed limit policy and zoning can be regarded as key instruments for more road safety on local and secondary roads.

Keywords: road safety, speed limits, Flanders Region, secondary roads, traffic signs database

1 Introduction

Assuring the ability to maintain high speeds can be seen as an important quality criterion of the mobility system: higher speeds reduce travel time and increase accessibility. But having higher speeds in road transport also has negative impacts, namely on road safety. Changes and differences in the traffic flow speed and speed differences between different vehicles also increase the risk of accidents. In addition there is the need for sufficient simplicity in the traffic tasks to be carried out by drivers - in particular because of the restrictions of the road user abilities. Concerning the vehicle characteristics, the mass and the speed together with the directions in which the traffic participants are moving, determine the degree of homogeneity. By means of a larger homogeneity the chances of conflicts in traffic can be reduced, so the more homogeneous the traffic flow, the more limited the chances of a conflict or accident. Because of the characteristics of both road users and vehicles, there is a need for uniformity in the traffic and it is not desirable to change speed limits or any change to the features of the road and its surroundings.

The Flanders region is located in the north of Belgium. Within the process of further decentralisation of political powers within the federal state structure, the regional Ministry of Mobility and Public Works has been obtaining more and more responsibilities for the mobility policy, including traffic regulation in the region from June 2014 on. On the secondary and local roads In Flanders special attention is needed for the 'environmental capacity' and for the safety of the most vulnerable road users. Indeed the spatial structure of Flanders is characterised by ribbon developments of housing and other buildings along these roads. Apart from the highly dispersed spatial urbanisation pattern, also the presence of schools and other functions along secondary roads, crossing towns and villages, attract a lot of people walking or biking along these roads or trying to cross the streets. Within this context a multitude of speed limits have been introduced on a step by step basis, each of the limits responding to a local concern, but without taking care of the overall continuity of speed patterns for car drivers that follow a certain route. In this paper we discuss the importance and opportunities to come to more homogeneous speed zones in the Flemish road network The paper builds on a research report [1] on the homogenization of speed zones, made by researchers from the Institute for Sustainable Mobility at the Ghent University.

2 Road safety as structuring principle for road network management and design

2.1 The road safety – speed relation

From road safety point of view establishing a speed limit is largely related to firstly avoiding the risk of an accident and, secondly, to limit the consequences of an accident, which could occur. In both cases, the type of vehicle is a major determinant. A loaded vehicle reacts differently when driving and braking. The severity of the accident is related to both the speed and the weight of the vehicle. For this reason, the relationship between general speed limits and specific speed limits in road segments (through additional regulations) and vehicle-related limits (e.g. 60 and 75 km/h respectively for trucks and buses on 1x2 roads in Belgium) are equally important. As related vehicle speeds differ greatly from the general speed limit on secondary and local roads, there is even more potential for conflict because of the lack of homogeneous speed in traffic flows on these types of roads. General as well as specific speed limits differ in European countries. In Belgium the general speed limit on 1x2 secondary and local roads in non-built up areas is 90 km/h, vehicle-related limits are 60 and 75 km/h respectively for trucks and buses. Because of a road safety concern, the Flemish Road Agency, responsible for the road management of regional roads, adopts as a general rule a lower limit (i.e. 70 km/h) than the one established in the national traffic regulation. On the other hand many local authorities stick to the 90 km/h limit. The general speed limit in Belgium in built-up areas is 50 km/h. As many secondary and local roads passing through town and village centres, on segments of these roads a 50 km/h limit is established.
Speed is generally accepted as a major factor in road accidents. Regardless of the exact circumstances of the accident, the speed factor is always present. In the study by Finch et al. [2], several European studies on the effect of increases and reductions in accidents at speeds are summarized. The researchers reached the conclusion that an increase in the average speed by 1 km/h results in an increase in the number of accidents by 3%, while a reduction of 1km/h leads to a decrease in the number of accidents by 3%. For more serious accidents even greater effects of the speed factor were found. Nilsson [3] presented the main results of the so-called 'power model' in which the speed-road safety is presented in a synthetic way, Figure 1.

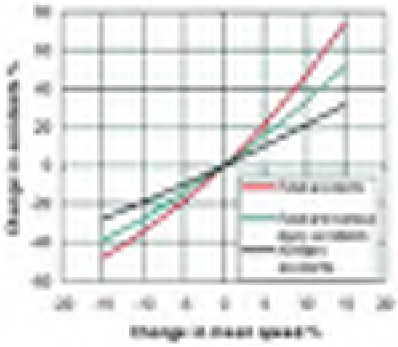

Figure 1 The power model of Nilsson: relation speed and traffic insecurity.

In a study by Taylor et al. [4] , the research by Finch et al. was refined. Taylor et al. note that in addition of a reduction of the mean speed in particular the reduction of the higher speeds (e.g. the number of drivers exceeding the speed limit) has an impact on the number of accidents. In addition, they found that the road type and the road environment are key regulatory factors for the effect of a speed reduction on road safety.

2.2 Road categorisation as a tool for road safety

Road categorisation is seen as a structuring factor for road safety. In the Netherlands, with its so called ‘sustainable road safety’ concept, one of the most advanced road categorisation systems in Europe is implemented [5]. The system relies on strict design standards per category in order to create ‘recognition’ of the road type by the road users (stimulating readability of driver tasks to be fulfilled), but also on functional use of the road system (e.g. long distance travel on the main and not on the local roads). The latter in order to contribute to a more homogeneous traffic flow.

The Flemish system of road categorisation on the contrary is a rather open system: it leaves space for interpretation of the general guidelines defined in the Flemish Spatial Structure Plan for different actors, as well for the selection of secondary and local roads by Provinces and Municipalities (resp. for the selection of secondary and local roads, based on the administrative EU-principle of subsidiarity), as for the road design (as there are no fixed design standards per road type). The advantage is that the road layout can be adapted to local spatial conditions (e.g. the design of road passages of secondary roads through town centres can take into account quality of life standards and traffic safety standards for pedestrians and cyclists). The disadvantage is that it may be contradictory to one of the main purposes of road categorisation: making the expected driver behaviour clearer to the road user by means of recognisable road types [6] [7].

This approach, with design guidelines based on the road categorisation and on the other hand the possibility to deviate from the guidelines due to local conditions and thus to a “partitioning “ of a road, leading to several successive segments with a different road layout (including those based on different design speeds), was developed in a study report on the design of secondary roads [8].

The compartmentalization is a principle that is based on the concept of the design of road passages through towns and villages used in Flanders since the eighties of the last century. It is based on a spatial and functional analysis of the road environment. By partitioning the road into segments, a speed differentiation and spatial differentiation of a road is obtained. However the intention remains to achieve the preferred situation, based on the general design guideline according to the road category. Such an approach is seen as a key to achieve road safety by tuning the road function, its actual use and its design to each other.

3 The Flemish Road Signs Database

The road sign database is an inventory of all traffic signs along the roads of Flanders . A total of ± 62,000 km of paved roads is inventoried, including ± 7,500 km along regional roads and ± 54,000 km along local roads. The Flemish road signs database contains several parameters: for the location of each sign , the x and y coordinates are tracked. Furthermore, there is information about the orientation (left or right side of the road), the street name and town, and the date of recording. Data of the type of traffic signs (according to the traffic signs classification in the national traffic regulation) are of course also inventoried with the dimensions of the plate and a picture of the set-up. Reference can also be made to the European project ROSATTE in which a methodology is developed to provide the data (relevant for road safety) to digital road map providers, used in GPS.
This road sign database can easily be turned into a speed management tool. It delivers the information on existing speed limits established all over the road network. Secondly this traffic signs database can be used to rationalize the location of the speed limit signs and guaranteeing safer traffic conditions by creating larger homogeneous speed zones. The concept of speed zones is the basis to rationalize the road management (as well for the road design as for the speed enforcement and also contributes to safer and easier driving. Traffic signs are an important source of information for the road user, determining the “conventions” and “rules” to behave in traffic. Moreover, the digital information of the speed limit signs can be converted into a digital speed map. The absence of a reliable digital speed map is one of the main technical barriers for the implementation of Intelligent Speed Adaptation (ISA). If vehicles are equipped with ISA and the system is implemented, not only a huge drop of the number and severity accidents can be achieved, but at the same time the driving task can be made considerably lighter.

4 Feasibility of more homogenous speed zones in Flanders, based on generic scenarios

The theoretical approach to the homogenization of speed zones can be tested in a case study on the Flemish secondary and local roads. In order to assess the consistency of the current speed limit policy in relation to the road functions as complete as possible for the Flemish region, it was decided to apply the choice of research areas according to the spatial classification of the different urbanisation typologies. The case study is limited to three types of analyses:

1. A first scenario examines what impact it can have on the number of road signs when the speed limit standard regime of 90 km/h in non-built up areas is reduced to 70 km/h. Main roads and primary roads for which the use of a higher speed limit than 70 km/h is believed , are excluded.
2. The second scenario analyses the length of the road segments along which a speed regime of 90 km/h is installed (as this is the standard regime, along this road segments there are no speed limit signs present) but that are surrounded by roads with a lower speed regime. In these cases it is examined whether the change in speed limit between these roads is justified.
3. In a third scenario an analysis is carried out of the extent to which the speed regimes correspond to the roads categorisation proposed in the local spatial structure plans.
4. These generic scenarios explore, within the limited nature of the cases, the possibilities offered by the road signs database to investigate the feasibility of a consistent speed limit policy.

4.1 Scenario 1: reduction standard regime 90 to 70 km/h

The assumption of this lower standard speed limit makes it unnecessary to provide speed signs type C43 (and C45 - end speed limit) and zone 70 signs (ZC43 and ZC45), as defined in the Belgian road signs code. It concerns all C43 and C45 speed limit signs along roads not located in built-up areas, where a lower speed regime is in force (zone 30, zone 50, ...), as well as roads that are not part of the road categories "Highways" or "Primary roads". Table 1 shows all the signs (S) C43 and (Z) C45 again with inscription 70 km/h. Under "Current signs" shall mean all signs (70 km/h), including those in built-up areas.

Table 1 Number of 70 km/h limit signs

TOWN	Number of signs (Z)C43 en (Z)C45 indicating 70 km/h					
	redundant signs (non built-up areas)			current signs (built-up and non built-up areas)		
	regional roads	local roads	total	regional roads	local roads	total
Lebbeke	19	55	64	19	55	74
Gent	99	70	169	166	70	236
Destelbergen	16	69	85	16	85	101
Wetteren	83	66	149	83	74	157
Aalst	59	1	60	59	1	60
Denderleeuw	20	2	22	28	2	30
Dendermonde	45	43	88	49	43	92
TOTAL	**341**	**306**	**637**	**420**	**330**	**750**

The redundant signs are each of the 70 km/h road signs, which might disappear if the standard speed limit regime is reduced to 70 km/h. This analysis does not account that possibly new speed signs should be added in order to install the 70 km/h speed regime. Further research could determine whether additional signs can be avoided. By applying zone indications systematically, a large number of road signs can be avoided. For a total of seven municipalities, changing the default regime to 70 km/h would make it possible to eliminate 637 signs, it is 85% of the total number of the currently installed 70km/h signs.

4.2 Scenario 2: Determining the length of reduced 90 to 70 km/h segments

The purpose of this analysis is to determine what the length of the segments with the same speed regime is after reducing isolated 90 km/h segments to 70 km/h. Two possibilities are considered:
1° continuous: segments along a single continuous path have different speeds. A road user goes straight ahead without having to carry out a manoeuvre or to turn left or right (see Figure 2).

Figure 2 Continuous road: 50-[90]-50

2° road changing on the intersection: in these cases, the road is adjacent to road with a different speed regime, being no continuous road (Figure 3) or it can concern a zone that one cannot leave without crossing a 90 km/h road.

Figure 3 Road changing via intersections: 50-[90]-50

The column "speed regime" in Tables 2 and 3 clarifies the maximum speed admitted on the adjacent road segments. The entry "70 - [90] -70" thus indicating a road where 70 km/h is allowed in a first segment, then 90 km/h and further on 70 km/h. The segment that caused the speed limit change is indicated between square brackets "[ ]", only for this segment, the average length is displayed in the tables. Zone30 regulations in school areas are not taken into account in this study because their large number and because of the absence of dynamic signs (30/50).

4.2.1 Continuous roads

In table 2 it can be seen that the speed limit changes along a continuous road usually occur in steps of 20 km/h. However, there are also roads where a speed limit change of 50 - [90] -50 occurs. The 50 km/h limit point in this cases roads passing through built-up areas. (this also applies to the more common 50 - [70] -50 combination). Speed limit changes of type 70 - [50] -70 indicate approaching intersections, and have an average length of 635 meters. Furthermore the transition segment to proceed from a standard speed regime 90 to 50 (for example, built-up area) can be implemented on the basis of a short section (on average 685 m) with a speed limit of 70 km/h (type 50 - [70] -90) areas.

Table 2 Length of road segments by speed limit regime

speed limit regime	number of segments	average length (m)	median length (m)
50-[90]-50	8	714.64	719.85
50-[90]-70	1	219.0	219.0
70-[90]-70	5	893.42	918.0
50-[70]-50	11	812.0	809.5
70-[50]-70	2	635	635
50-[70]-90	4	685.1	571.6
70-[50]-30	2	444.1	444.1
TOTAL	**33**	**737.4**	**695.9**

The average speed limit change runs over a length of 737 meter. If one takes into account the time which the vehicle requires to slow down or to accelerate, the higher speed limits on such short distances, can be called into question.

4.2.2 Road changing via intersections

In Table 3, the results of this part of the scenario are displayed. The individual road segments of 90 km/h roads adjacent to a lower speed road are at average 682 meters long. These are usually rural roads that fall outside a specific speed limitation indication.

Table 3 Length of road segments by speed limit regime

speed regime	number of segments	average length (m)	median length (m)
50-[90]-50	5	714.4	603.1
50-[90]-70	5	570.648	711.5
70-[90]-70	4	895.025	1044.9
30-[90]-50	1	227.6	227.6
TOTAL	**15**	**682.1**	**711.5**
speed regime	**number of zones**	**average length summed length road segments in zone (m)**	**median length summed length road segments in zone (m)**
[90] omsloten door 70 en 50	1	4674.9	4674.9
[90] omsloten door 70	4	3634.9	3734.7
[90] omsloten door 50	3	2420.6	3072.7
TOTAL	**8**	**3309.5**	**3072.7**

In some cases also groups of roads(zones) surrounded by roads with a lower speed occur. This cases usually also consist of local country roads or forest roads, where no specific speed regime is applied, and therefore the speed limit is 90 km/h. In total, this applies in eight areas with an average total length of roads is 3.3 kilometres.

4.2.3 Scenario 3: Road categorisation speed limits implementation

The extent to which the speed limits currently applied coincide with the desirable limits according to road classification was examined for two municipalities (Wetteren, Destelbergen), for these municipalities the current road categorization is known and shown in the local mobility plans. The effective speed limit which is displayed in the traffic signs database, and hence on the field, can be either equal to, greater than or less than the preferred limit according to the road categorisation standards. The most desirable is, of course, an equal speed limit. When the preferred speed is less than the actual speed, it does not rhyme with the desired function of the road and road safety is compromised. It is also possible that the preferred limit is higher than the actual speed limit. In that case, the comparison must be of the preferred speed with effective speed approached with caution, since the possibility of the introduction of the partitioning of the road based on the vulnerability or the surrounding functions and activities is not taken into account in the standard.

Table 4 shows for 'Main roads' and 'Primary roads I' a continuing compliance with the guidelines regarding the preferred speed limits. For roads within the category of 'Primary road type II' abnormalities are well established. For 27% of the Primary II roads the preferred limit is higher than the actual speed limit. It comes to roads not equipped with a physical traffic flow separator, so that the implemented speed limit of 70 km/h is indeed justified. For 'Secondary roads' in all studied cases the applied speed limit complies with the preferred limit according the road category. In contrast, the practice differs sharply for 'Local roads Type III', this type shows the largest share of deviation (48%). This is due to the standard regime speed of 90 km/h, which is in force when no specific speed limit sign is installed. On the other hand derogations in which the effective speed is lower, occur (in 14 % of the cases). This is due to the assignment of the 30 km/h speed limit instrument. This partitioning is often required in school environments or other vulnerable areas, and can hardly be labelled as undesirable.

Table 4 Comparing preferable speed limit according road categorisation and current limit

Road-category	preferable limit = current limit	preferable limit › current limit		preferable limit ‹ current limit		Total (km)
	Total road length (km)	Total road length (km))	Average deviation speed limit (km/h)	Total road length (km)	Average deviation speed limit (km/h)	
Main road	15,3	–	–	–	–	15,3
Primary I	10,4	–	–	–	–	10,4
Primary II	2,2	2,8	-23	5,6	+10	10,7
Secondary I	0	–	–	–	–	0
Secondary II	9,8	–	–	–	–	9,8
Secondary III	1,3	–	–	–	–	1,3
Local I	20,1	2,2	-20	2,9	+20	25,2
Local II	21,3	3,8	-25	4,2	+23	29,3
Local III	96,9	35,5	-20	124,4	+38,5	256,8

5 Conclusions

Both the speed level and the (lack of) of homogeneity in the traffic flow are major road accidents factors. Belgium applies outside the vehicle-related speed limits, also limits which are related to the type of road and which are determined by the responsible road authorities (regional or local). Determining permitted speeds on 2x1 roads outside built-up areas either within the framework of the existing general speed limits (90 km/h) or in the context of any newly introduced overall lower speed limit level (70 km/h), should be assessed on its effects on road safety, driving comfort and traffic flowing.

On the basis of seven cases, spread over different spatial types of municipalities, the effect of decreasing the overall speed limit of 90 to 70 km/h was studied. Although the use of generic scenarios (much effect may be expected on the implementation of speed regime zone road signs) depend on the application of the zone, using the road signs data base offer only a first, rough analysis, it can be concluded that, in the lowering of the speed limit of 90 to 70 km/h on local and secondary roads in Flanders, 6 of every 7 speed limits signs can be removed. This does not only mean that the homogeneity of the traffic is going to increase on these roads but it can also reduce the workload for the driver and the costs for the road authorities, as the length with the same speed limit would increase substantially.

The case studies therefore indicate that the deployment of this scenario over the entire road network in Flanders, supported by the use of the Road Signs Database, offers an interesting research track to assess more in depth.

References

[1] De Mol, J., Lauwers, D., De Baets, K., Allaert, G.: Homogeniseren van snelheidszones, Steunpunt Mobiliteit en Openbare Werken Spoor Verkeersveiligheid, Instituut voor Duurzame Mobiliteit, Universiteit Gent, Gent, 2011.

[2] Finch, D., Kompner, P., Lockwood, C., Maycock, G.: Speed, speed limits and accidents. Project Report PR58. Transport Research Laboratory, Crowthorne, 1994.

[3] Nilsson, G.: Traffic safety dimensions and the power model to describe the effect of speed on safety. Lund Institute of Technology, Lund, 2004.

[4] Taylor, M., Lynam, D., Baruya, A.: The effects of drivers' speed on the frequency of road accidents, TRL Report, No. 421, Transport Research Laboratory TRL, Crowthorne, Berkshire, 2000.

[5] S.n.: Handboek wegontwerp, CROW, Ede, 2002.

[6] Lauwers, D., Gillis, D.: Towards new approaches of road networks, integrating transport and land use objectives - cases in Europe, International Road Federation World Meeting, Technical & Scientific Program, Riyadh, 2013.

[7] Lauwers, D., Gillis, D.: Towards new principles of road categorization: reflections based on practices in Belgium and Eastern Europe, 1st International Conference on Road and Rail Infrastructure – CETRA 2010, 17–18 May 2010, Opatija, Croatia, pp. 231-237.

[8] Engels, D., Devriendt, K., Lauwers, D.: Handboek secundaire wegen, Tritel and Iris consulting, Brussels, 2003.

SAFETY MEASURES IN ROAD TUNNELS

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Abstract

Road tunnels today, because of their technological and traffic characteristics, belong to the group of highly complex structures. Tunnel safety is primarily a social and then an economic category. It is determined by the relations between road users, vehicles, transportation infrastructure and other factors in its dynamic system. Fire and other major accidents that can occur in road tunnels not only cause direct costs due to the resulting damages and closure, but also threaten human communities and markets related to tunnels. Therefore, it is evident that a high level of safety in tunnels provides a wide range of benefits. This paper will analyse basic safety elements that should be taken into account when designing road tunnels. It will provide an overview of the basic traffic safety elements which are implemented in road tunnels in Croatia.

Keywords: road tunnels, traffic safety, tunnel design

1 Introduction

Tunnels are underground structures that must, in a safe manner, enable traffic to people and goods between two geographic points on Earth, divided by natural or man-made obstacles. The increase in traffic leads to a growth in risks from accidents in tunnels. Therefore, it is necessary to raise the level of traffic safety to an adequate and socially acceptable level by installing various systems that enable traffic safety. Accidents in tunnels often cause terrible consequences for road users, whereby even a minor incident can result in significant material damage and loss of human lives. Therefore, utmost care should be taken when equipping tunnels with systems such as power supply, lighting supply, ventilation and other safety systems that are fundamental elements for establishing high-level safety required in tunnels. This paper will present measures necessary to prevent accidents and to establish the highest level of safety in road tunnels.

2 Minimum safety requirements for tunnels

2.1 Minimum safety requirements for tunnels according to the EU Directive

The regulation that defines the minimum safety requirements for tunnels as engineering structures is the "Ordinance on minimum safety requirements for tunnels" [1], being the Croatian national regulation, which actually represents alignment of the Croatian legal system with the EU Directive [2].
The EU Directive [2] was issued at the level of the European Union with the objective of aligning the minimum requirements of the national rules and regulations of the member countries, with the aim of raising and harmonising the level of safety in European road tunnels. Although its provisions are formally only applied to trans-European road tunnels over 500 m

long, they are often applied to tunnels outside this network as a minimum safety criterion. When examining the level of safety in tunnels, we examine factors that can be classified into four main groups, as shown in Figure 1. The stated factors affect safety both individually, as well as in their interaction. Taking into consideration their interplay, it can be concluded that traffic safety as a whole is an extremely complex issue. The Croatian safety system, introduced on the basis of the EU Directive [2], has set forth the following functions/institutions for road tunnels:

- Administrative Authority;
- Tunnel Manager;
- Safety Officer;
- Inspection Entity;
- Emergency Services.

An important element of tunnel safety is the safety documentation, which must contain descriptions of all prevention and safety measures, as well as information essential for safety.

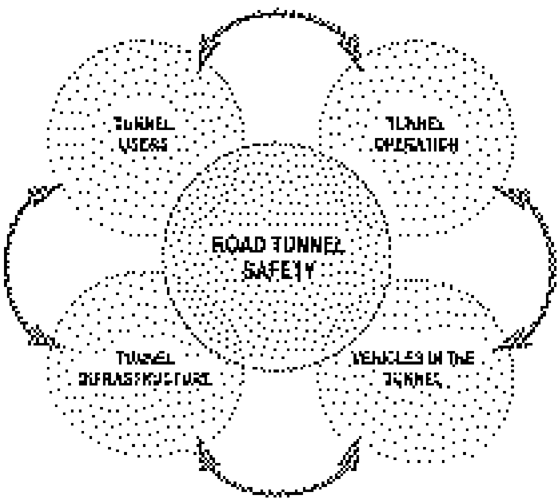

Figure 1 Safety factors in tunnels

2.2 Traffic signs and traffic safety

Factors that affect road traffic safety in general are: road route, technical road elements, condition of roadway, road equipment, lighting, intersections and road maintenance system. In global analyses of road traffic accidents, the following factors are always taken into account: man as road user, vehicle, road, traffic environment and incident factors.
Road traffic encompasses:

- Traffic organisation (traffic regulations and technical means for traffic organisation);
- Traffic management;
- Traffic control (monitoring traffic flow and volume, intervention in emergencies, recording and analysing traffic accident statistics).

Traffic signs are one of the characteristics of roads. The basic role of traffic signs is to promote safety and to successfully manage traffic flow. Traffic signs are used to inform road users on the condition of the road, allowing safe and unhindered traffic flow. Traffic signs must be simple, clear, understandable, visible, unambiguous, universal, continuous, appropriately designed and placed in sufficient quantity (Figure 2.).

2.3 Weather conditions and traffic safety

Weather conditions in traffic, as well as properly observing and measuring them, exhibit a significant influence on the safety of road users. When analysing weather conditions, the following data are measured: air temperature, roadway temperature, roadway humidity, roadway state, presence of salt and chemicals, intensity and quantity of different types of precipitation, air pressure and humidity, wind speed and direction, snow height, degree of iciness, freezing point, visibility and road illumination.

Figure 2 Example of properly installed traffic signs in a tunnel [3]

Obtaining reliable meteorological data is a complex process which is essential for road traffic safety. Reliable and accurate information is crucial both to end users of roads, as well as to road maintenance services for the purpose of road safety and maintenance.

2.4 Procedure in case of emergency in tunnels

An important safety factor refers to the procedure in case of emergency, defined in the instructions on proper steps to be taken in case of emergency in tunnels. Incidents can occur at any moment, while the proper steps to be taken can prevent greater damage and enable safe flow of traffic for other road users. The instructions refer to basic steps in case of the following incidents:
- Traffic hold-up in tunnel;
- Vehicle breakdown in tunnel;
- Traffic accident in tunnel;
- Fire in tunnel.

3 Euro Tap project

3.1 EuroTAP project methodology

EuroTAP (European Tunnel Assessment Programme) [4] is one of eight research projects on tunnel safety and is directly related to raising the level of traffic road safety. It was established on the basis of European Directive 2004/54/EC [2]. The EuroTAP project includes 11 automobile clubs from 10 European countries. The testing of each tunnel is conducted through the following steps [5]:
- Meeting with tunnel operator and collecting basic information on the tunnel;
- Driving through the tunnel in the presence of the tunnel operator;
- Inspection of characteristic locations;
- Random control of safety equipment;
- Taking photos of the tunnel;
- Inspection of the tunnel control centre.

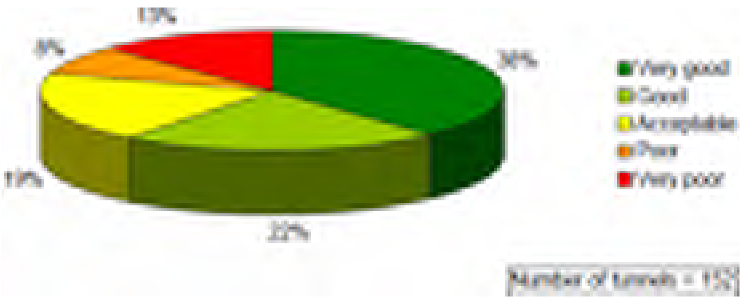

Figure 3 Total result for the years 2005 to 2007 [5]

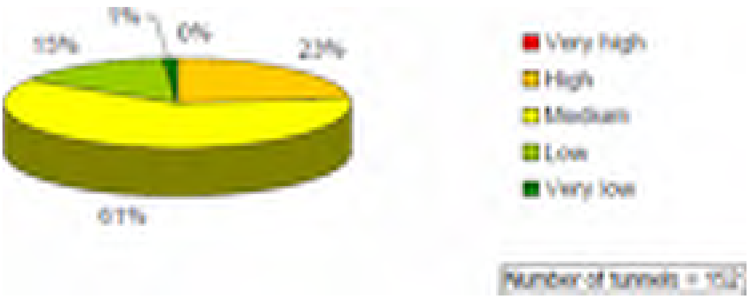

Figure 4 Distribution of the risk potential for the 2005 to 2007 period [5]

3.2 EuroTAP project results in the period from 2005 – 2007

In the period from 2005 to 2007, the safety of 152 tunnels was inspected as part of the EuroTAP project. The results show that most of the tested tunnels received a positive grade (Figure 3.). The grade distribution, presented in Figure 3. shows that a surprising 21% of tunnels, that is, every fifth tunnel, received a negative grade (poor and very poor). The main inadequacies in the individual categories, that is, the areas that received the poorest grades are the following:
· Non-existence of loudspeakers;
· Insufficient number of lanes;
· Low level of illumination;
· Non-existence of information signs on portals;
· Non-existence of evacuation lighting and signs in case of emergency;
· Lack of appropriate fire protection equipment for protecting the respiratory tract;
· Lack of equipment for inspecting longitudinal air flow;
· Irregular conducting of safety drills;

By examining the distribution of risk potential for the tunnels tested in the observed time period, it is evident that a surprising 23% of tunnels had a high risk potential, as can be seen in Figure 4.

4 Assessment of road tunnel safety

The safety assessment encompasses all safety measures that have to be undertaken and that refer to tunnel structure, technical equipment and tunnel organisation. It is divided into eight categories, as shown in Table 1. The linking of influence factors and safety measures that should be analysed and applied when planning and designing new tunnel systems is provided in Table 2 and Table 3.

Table 1 Review of safety measures [1, 2, 5]

Category of safety measures	Safety measures
Tunnel system	· Number of tubes; · Width and layout of traffic lanes; · Geometry and layout of emergency lanes/lay-bys; · Geometry and layout of emergency walk-ways; · Brightness of tunnel walls; · Additional measures (portal design, road surface, tunnel route).
Lighting and power supply	· Lighting throughout and adaptation zones; · Power supply (utility and internal); · Emergency power supply.
Traffic and traffic surveillance	· Congestion in the tunnel; · Speed limits; · Control centre; · Restrictions for and/or registration of vehicles carrying hazardous goods; · Automatic detection of traffic and congestion; · Video surveillance; · Traffic control (traffic lights, variable traffic signs, signs, etc.); · Measures to close the tunnel (traffic lights, barriers, information displays); · Traffic signs; · Visual guidance equipment; · Additional measures (e.g. for heavy goods traffic, monitoring the distance between vehicles and speed, automatic recognition of hazardous goods traffic, height detectors).
Communication	· Loudspeakers; · Traffic radio; · Emergency phones (distance, signs, functions, insulation against traffic noise); · Tunnel radio.
Escape and rescue routes	· Distance between emergency exits; · Emergency exit signs; · Prevention of smoke from penetrating escape routes, fire rating of doors; · Evacuation lighting and escape route signs in the tunnel; · External access for fire and rescue services; · Access routes for fire and rescue services; · Additional measures (special lighting for emergency exits, signs showing what to do, barrier-free emergency exits).
Fire protection	· Fire protection of the tunnel structure; · Fire resistant cables; · Fire alarm systems (automatic/manual); · Extinguishing systems (arrangement, signs, functions); · Drainage system (system for draining flammable or toxic liquids); · The time it takes for the fire brigade to arrive; · Fire brigade training and equipment.
Ventilation	· Ventilation in normal mode to thin out vehicle emissions; · Special fire programmes; · Control of the longitudinal flow in the tunnel and consideration of this in ventilation control; · Temperature stability of facilities and equipment; · Proof of correct functioning in fire trials and by flow measurements; · Longitudinal ventilation (Air flow rate, Length of ventilation sectors, Air flow in the direction of traffic, Reversible fans); · Transverse/semi-transverse ventilation (Volume flow of extraction, Capacity to control longitudinal flow, Opening/closing of the exhaust air outlets can be controlled).
Emergency management	· Emergency response plans; · Automatic linking of the systems; · Measures in the case of an accident or fire; · Regular emergency drills; · Regular training for tunnel control centre staff; · Maintenance plan.

Table 2 Possible evaluation of influence factors [5]

	HAZARD POTENTIAL				
INFLUENCE FACTORS	**VERY LOW**	**LOW**	**MEDIUM**	**HIGH**	**VERY HIGH**
Length of tunnel [m]	500 – 1.000	1.000 – 1.500	1.500 – 3.000	3.000 – 5.000	› 5.000
Volume of traffic [vehicles/day/lane]	‹ 2.000	2.000 – 5.000	5.000 – 10.000	10.000 – 15.000	› 15.000
HGV percentage [HGV mileage/day/tube]	‹ 500	500 – 2.000	2.000 – 6.000	6.000 – 12.000	› 12.000
Hazardous goods traffic [No. of HGVs/day]	‹ 10	10 – 50	50 – 300	300 – 1.000	› 1.000
Gradient in the tunnel [%]	‹ 1	1 – 3	3 – 5	5 – 7	› 7
Speed [km/h]	‹ 50	50 - 70	70 - 90	90 - 120	› 120
Access time for emergency services [min]	‹ 5	5 -170	10 - 15	15 - 20	› 120

Table 3 Linking influence factors and safety measures [5]

SAFETY MEASURES	LENGTH	Volume of traffic	Percentage of HGVs	Traffic routing	Hazardous goods	Speed
I.Prevention						
Number of tubes		•		•		
Lane width			•	•		•
Lay-bys	•	•				
Lighting in the tunnel	•	•	•	•		•
Video surveillance	•	•			•	
II.Detection						
Incident detection	•	•			•	
Fire alarm system	•	•				
SOS phones		•				
III.Self-rescue						
Emergency exits	•	•				
Ventilation system	•	•		•		
IV.Incident management						
Barriers/information displays to close the tunnel	•	•				•
Tunnel radio	•	•				
Rescue routes for emergency service vehicles	•			•		

5 Characteristics of tunnels on Croatian motorways

Croatia has a total of 42 kilometres of tunnels on motorways, of which 11 are over 1,000 m long. The longest Croatian tunnels are Mala Kapela, Sveti Rok, Učka, Sveti Ilija, Plasina, Tuhobić, which are over 2,000 m long, while Učka is the only tunnel consisting of only one tube without an additional service tube [6]. Concerning safety, Croatian tunnels maintain a high standard, which is supported by the EuroTAP project results [4]. In 2005, Plasina tunnel took third place. In 2006, Grič tunnel was ranked second, whereas in 2007, Brinje tunnel was declared the safest tunnel in Europe. The main positive characteristics of Croatian tunnels on motorways are evident from the following [6]:

- They are built as dual-tube or single-tube with an additional service tube;
- Complete video surveillance;
- Lay-bys and connections to the other tube;
- SOS telephone system installed and fire extinguishers available;
- Larger tunnels are managed from traffic control centres.

6 Conclusion

Tunnels, as road structures, represent an important element of traffic infrastructure, in particular of motorway networks. Longer traffic hold-ups due to accidents are not acceptable, let alone human casualties that can be avoided by applying contemporary safety systems and measures. The EuroTAP tunnel test is a key factor for introducing measures defined by EU Directive (2004/54/EC) and national regulations of individual member countries, which should be taken into consideration already in the design phase in order to raise safety levels in tunnels. Analysing the tunnel network in the Republic of Croatia, it can be concluded that the tunnels possess a satisfactory level of safety. They are equipped with state-of-the-art fire protection and fire alarms, dynamic signs and video surveillance systems, as well as other control and management systems that significantly contribute to raising the level of safety in tunnels.

References

[1] Pravilnik o minimalnim sigurnosnim zahtjevima za tunele, NN 96/13, Zagreb, 2013.

[2] EU Directive (2004): Directive 2004/54/EC of the European parliament and of the Council of 29 April 2004 on minimum safety requirements for tunnels in the trans-European road network, Official Journal of the European Union L 167, 2004.

[3] Bettelini, M.: Sanierung Montblanc-Tunnel, Tec21, 127 (2001), 7-11

[4] EuroTAP (European Tunnel Assessment Programme). http://www.eurotestmobility.eu/

[5] EuroTap, Tunnel Audit, final report, Munich, 2007.

[6] Komić, I.: Diplomski rad – Sigurnost u cestovnim tunelima, Zagreb 2013.

APPROACHES TO SOLVE THE PROBLEM OF PASSIVE SAFETY OF PASSENGER WAGONS

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Abstract

Railway safety is concerned with the protection of life and property through regulation, management and technology development of all forms of rail transportation. The analytical results of the world modern tendencies and approaches to solve the problem of a passive safety of railway vehicles, in particular passenger wagons are presented. Also, the existing legislation of the European countries that regulates a passive safety of express and high-speed passenger trains is considered. The basic concepts of the design of a passive protection of a high-speed passenger train wagon are determined. The paper analyzes the necessary reconstruction of passenger wagons in Bulgaria to ensure their passive safety.

Keywords: rolling stock, railway maintenance, innovation, vehicle dynamic, passenger wagons (coaches), passive safety

1 Introduction

The problems of passive safety of passenger wagons (coaches) are treated in regard to compliance with the regulations of the EU, mainly with the Technical Specification for Interoperability (TSI) relating to the rolling stock subsystem – "Locomotives and passenger rolling stock" of the trans-European conventional rail system (notified under document C (2011) 2737) [1]. The TSI is applicable to all units except for the units that are not designed to carry passengers or staff during operation and OTMs "On track machines", which are vehicles designed especially for construction and maintenance of track and infrastructure. Furthermore, the units that cannot reach speeds specified in any of the collision scenarios given below are excluded from the provisions related to that collision scenario. The passive safety measures are intended to be supplement to the active safety ones when all other measures have been exhausted. For this purpose, the mechanical structure of vehicles has to meet the requirements for construction of railway vehicle bodies [2] and ensure the protection to people in case of a collision by providing means for:
- deceleration limiting;
- keeping the survival space and structural integrity of the residential premises;
- reducing the risk of getting the wagons one on another;
- reducing the risk of derailment;
- limiting the consequences of hitting an obstruction on the track.

In order to keep these functional requirements, the units must comply with the detailed requirements set in EN15227: 2008 + A1: 2010 standard [3] to crashworthiness design category CI (according to Table 1 of EN15227: 2008 Section 4), unless something different is stated.

The following four basic collision scenarios will be examined:
· Scenario 1: a front end impact between two identical train units;
· Scenario 2: a front end impact with a different type of railway vehicle (with a freight wagon);
· Scenario 3: a train unit front end impact with a large road vehicle on a level crossing;
· Scenario 4: a train unit impact into a low obstacle (e.g. car on a level crossing, an animal, a piece of rock, etc.).

These scenarios are described in Table 2 in Section 5 of EN15227: 2008 standard. Within the scope of the current TSI, the rules in Table 2 are supplemented with the following:
· The application of the requirements related to scenarios 1 and 2 for heavy haul locomotives used only for freight operations and fitted with central couplers that comply with the principle of Wilson (e.g. SA3) or Jenny (AAR standard) intended for operation on CR TEN track is an open question;
· The conformity assessment of central-cab locomotives with the requirements of Scenario 3 is still an open question.

The current TSI defines the crashworthiness requirements applicable within its scope. Therefore Annex A to EN 15227:2008 standard is not applied. The requirements of Section 6 of EN15227: 2008 standard shall be applied in regard to the reference collision scenarios mentioned above.

2 State-of-the-art in the Bulgarian railways

If we trace the period of 50-60 years backward under the condition of the Bulgarian railways (BDZ), it can be seen that front end impact accidents with fatalities have a clear tendency to decrease. For instance, while for the period of 60 years until now the fatalities caused by front end impacts were over 70, for the past 10 years only one man died – the driver of the passenger train who did not stop at a red signal and crashed into the stopped fast train. The favourable tendency mentioned above is primarily due to duplication and ALS introduction on main tracks, the increase of train braking security as well as to improvements in passenger rolling stock, namely to the substitution of the old-structure coaches (mostly two-axle and three-axle ones) with new four-axle coaches with structures entirely made of metal. The latter meet the requirements of UIC for extra load bearing. Considering the decreased number of collisions, it should be also mentioned the influence of significantly reduced volume of rail freight in the past two decades. Speaking about the front end impact scenarios including No 1 and No 2, one should not neglect the lateral collisions with locomotives or wagons: there were 25 casualties due to such accidents in the period of 25 years but none for the past 10 years. At present as well as in earlier periods the most significant accidents (in number of accidents and number of fatalities) are those that happen at level crossings: about 70 people were killed for the last 35 years and 29 died for five years (2009 -2013). Most of them were in road vehicles and pedestrians. In 2012 there were 30 accidents at level crossings with 7 fatalities and 15 wounded (Table 1). As it can be seen, the increase of this type of accidents is a clear and disturbing tendency. The main reason is indiscipline, aggressiveness, negligence even to the own lives of drivers. Moreover, this disturbing tendency has been increasing regardless of the increasing number of automated level crossings (with electric barriers with or without level-crossing keepers in the area of stations and level crossings with automatic signaling). In Bulgaria only about 22% of all level crossings are with manually-operated barriers and 17% are without any facilities, i.e. only with signs. At that, it is curious and disturbing at the same time that the official statistics show the lowest number of accidents at these particular level crossings, which have only signs (but it is apparently because the frequency of trains and motor vehicles is not considered). It is also interesting that Bulgaria occupies the top position in the EU by the indicator "equipment of level crossings" but contrary to expectations, the country is on the top places also by both the number of level crossing accidents and the number of deaths caused as a result of those accidents.

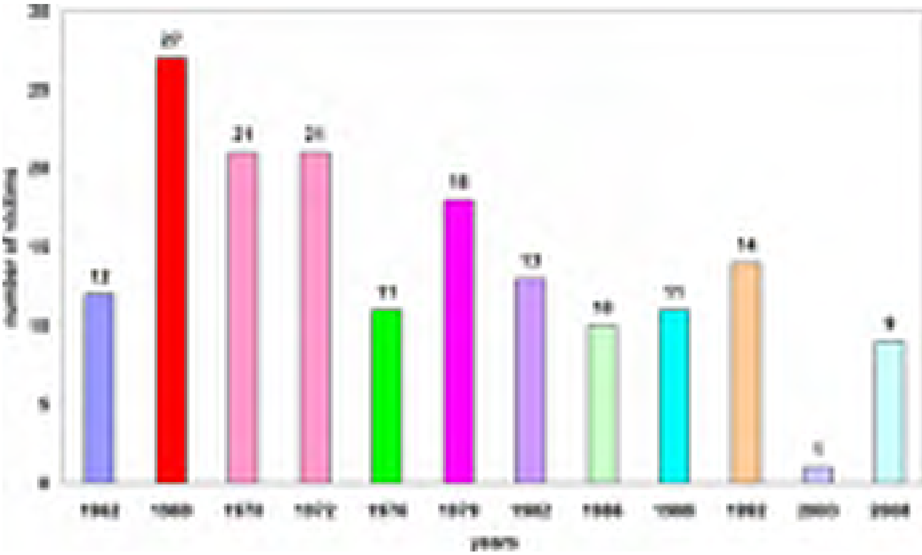

Figure 1 Number of fatalities due to railway accidents in Bulgaria since 1962 (according to the data taken from [4])

Table 1 Number of deaths and injured in 2012 due to railway accidents [4]

Indicators	Accidents	Deaths	Injured
Collision	5	1	5
Derailment	3	-	-
Accidents on level crossings	30	7	15

Of all railway accidents (heavy and light) until now, those with derailment happened most often (over 90% of the total number). Almost all cases of derailment were with freight wagons and locomotives, at low speeds and usually while shunting. However, there were two cases of derailment (in 1974 and 1980) of fast passenger trains at a speed of 90 km/h caused by the loss of stability on the track at high temperature. Although with these accidents the wagons had lost the connection with each other and were scattered over a distances of 50-100 meters and even farther from the track, many of them were turned on their sides (at 90°) and “on their back”, yet their integrity was preserved and the number of fatalities was minimal (one passenger). Apparently, that could be explained by the fact that the wagons were newly-constructed, had passed through all kinds of tests including those under emergency loads with longitudinal and vertical forces as required by the UIC. There are certain reasons to claim that if the wagons had been of old types, the number of passengers killed in such severe crashes would have been at least 10 times bigger.
It is worth analyzing two more severe crashes (in1969 and 1980): the first one happened to a passenger wagon with many casualties (27 fatalities and 38 seriously injured many of whom consequently died); the second one was with a freight train without fatalities. The first crash (the one of a coach) represents a typical case of extra-loading with passengers and luggage, which caused loss of stability, i.e. violation of the “iron” condition: the metacentric height should be greater than the height of the center of gravity. In fact this led to overturning the wagon side (на 90°) and towing by the locomotive in this state about 250 m where many passengers fell through window openings, which lead to a fatal end. It is also important to note that the overturned wagon was actually a narrow-gauge 760 mm utility cart of Fiat trainset known for its high comfort at the time being: modern furniture, big windows and “soft”

spring suspension, i.e. with springs of large static deflection - f_{st}. Obviously, the overload of wagon resulted in even greater static deflection, hence reduced the metacentre height further (because it is inversely proportional to f_{st}), and on the other hand, to increasing the height of centre of gravity. Thus with divergent changes of these two heights (of metacenter and centre of gravity), they reached their equality, i.e. violation of the above mentioned condition, depletion of wagon stability with fatal consequences. BDZ has accepted that the newly-built passenger wagons should be with advanced features of springs. For example, the coaches produced in1987 with static deflection of more than 300 mm have rubber pads inserted into the springs of the central level that come into action at high load when sustainability could worsen without them. Of course, it is on the expense of worsened smoothness of running but taking into account that such cases arise quite rarely, the solution in favour of safety is preferred.

The second accident of the ones mentioned above (with a freight train) was caused by a loss of overturning resistance due to large centrifugal force in a curve as a result of speeding. In this case the freight train was "released" in an area of steep slopes and sharp curves, so due to the terrain peculiarities the wagons slid (or rolled) to a deep ravine.

In 2008 there was a fire accident with 9 fatalities. The most important conclusion based on that and other similar accidents (but without fatalities) is that it is necessary to unconditionally meet the requirements for fire-resistance of materials used in wagons furniture.

The statistics of the Bulgarian railways shows that the accidents with derailment are the most frequent ones. The main reasons for derailment are the defects on the track and violation of the signs requiring deceleration. Speeding is a consequence of train driver's errors as well as of vehicles moving uncontrollably on the track (the so-called "released" vehicles). The activities undertaken in regard to derailment belong to the group of active safety.

Taking into account the experience of the Bulgarian railways and international assessments, it is seen that railway collision accidents are still significant due to frequency and damages caused to passengers. However, even with the assumption that the frequency of collisions will be significantly reduced in future but considering the tendency of speed and damage increase with the square grade of speed, it should be concluded that accidents of front end impacts are the most serious problem.

3 Approaches to technical solutions

To solve the problem of passive safety with railway collision accidents, the technical solutions applied can be classified into two groups: active – aimed at preventing the occurrence of emergencies and passive – intended to reduce the possible negative consequences of actual accidents. It is because the analysis of international experience has shown that even the application of all possible means of active safety does not make possible to completely avoid emergency involving death and injury of passengers. Therefore the development of systems and devices to provide passive or structural safety of wagons can be assessed as a priority trend in passenger transport. To minimize the negative effects of emergency collisions is carried out by embedding the so-called crash systems to the bearing structure of the body. These systems contain devices for absorbing the energy of a strong singular impact (this concerns the impact that requires energy-absorbing more than the available We0, realized by elastic deformation of the buffer energy absorbing devices.) that emerges random by destruction with irreversible plastic deformation. These destructing devices and components, also known with the definition of "victims", should meet the following conditions:

- simplified structure;
- small mass;
- low cost;
- easy and convenient replacement after being destroyed.

Due to economic considerations, it is appropriate to build crash systems at least in two stages: First degree: with devices (elements) embedded in crash absorbers (i.e. buffers or automatic couplings), which according to the current possibilities may have energy-absorbing capacity of We ≥ 0,8 MJ1). In compliance with the standards, which are in force for in-service modern wagons and newly-constructed ones, the maximum force should be of about 3,2 ÷ 3,6 MN (pair number buffers).Second stage: with devices located in the front (transition) parts of the wagons where there might stand only people occasionally passing between them. The minimum value of force, at which the second level becomes active (in order to preserve it in lighter strokes of power up to 3 MN (this value of strength corresponds to the minimum value of the safety coefficient of static loads as well as to the standards of crash loads, which should not cause inadmissible plastic deformations.) and kinetic energy to ≤ We0 + WeI), can exceed the strength of the first stage with mean 10% (4 ÷ 4,4 MN). The probability of its storage is at least 70 %. On the other hand, this power shall not exceed the threshold, at which inadmissible plastic deformations of the metal structure could occur in the part occupied by passengers. Provided that the safety deformation of the second degree in the front part of the vehicle can not be more than “1m and considering the requirement of power (4 ÷ 4,4 MN), it is seen that energy absorption can not exceed 4,4 MJ for the one side of the wagon.

The following requirements have to be met additionally:

1 The second stage of the crash system, which according to its purpose should be destroyed and replaced, must be connected to the main structure by bolted joints;
2 To determine the boundary conditions of collision (by speed, type and number of vehicles, etc.) where:
   a derailment of some wagons can occur;
   b the longitudinal acceleration must not exceed the permissible value for passengers (5g);
   c the permissible value of the longitudinal acceleration (e.g. the value of 5g, which is assumed as permissible) is precised at what degree of probability preserves life or health of passengers.
3 The existing legal framework in the European countries, which regulates passive safety, shall be constantly updated and adapted to increase the passive safety of passenger wagons.

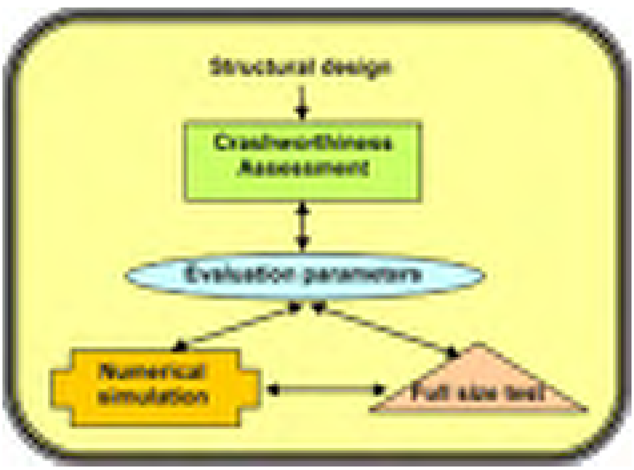

Figure 2 Fig. 2. Elements of the design process of a crashworthiness structure

Figure 3 Fig.3. Crash buffer of a sleeping carriage operated in Bulgaria since April 2013

The design of passenger wagons is an iterative process between the process of design, selection and evaluation of a crashworthiness structure and the assessment methods. Fig. 2 shows the elements of the design process of the crash-resistant structure. The existing evaluation methods, criteria or standards are two: numerical simulation and full tests [5]. The parameters of evaluation are: deformation, energy absorption, material properties, non-linear limitations and contact. Based on the optimized parameters, an algorithm and solution as well as possibilities for structural simplification have been proposed. The crashworthiness structure of a passenger wagon frame is a subject of study carried out at the Todor Kableshkov University of Transport, in Sofia, Republic of Bulgaria. Fig. 3 shows the crash buffer of a sleeping carriage operated in Bulgaria since April 2013.

4 Conclusion

Based on the current regulations in the EU and the global developments in the construction and operation of passenger wagons (coaches) as well as the overall trend of speed increase, it can be concluded that the current priority in passenger traffic is to increase their passive safety by building crash-systems at least in two stages taking into account the established permissible longitudinal loads for in-service passenger wagons with modern structures and the newly-built ones. The main parameters (force and approximate energy absorption) of the 1st and 2nd crash systems of passenger wagons are also determined. A number of questions, which are "hanging" at present but should be made clearer on the basis of consensus and/or research, are also marked.

References

[1] COMMISSION DECISION of 26 April 2011 concerning a technical specification for interoperability relating to the rolling stock subsystem — 'Locomotives and passenger rolling stock' of the trans-European conventional rail system (notified under document C(2011) 2737)

[2] EN 12663-1:2010: Railway applications - Structural requirements of railway vehicle bodies - Part 1: Locomotives and passenger rolling stock (and alternative method for freight wagons)

[3] EN 15227:2008+A1:2010: Railway applications – Crashworthiness requirements for railway vehicle bodies

[4] National Statistical Institute, Republic of Bulgaria, http://www.nsi.bg

[5] Petrovic, D.: Dinamika sudara vagona, Biblioteka Dissertacio, Zaduzbina Andrejevic, Beograd, 2001

FACTORS INFLUENCING DRIVER'S BEHAVIOUR AT INTERSECTIONS CROSSED BY THE TRAM

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Abstract

Drivers' behaviour in intersections crossed by the tram is not only influenced by the rules of the tramway priority in the intersection but also by the road features that influence right or wrong behaviour of the driver. The driver follows the road with expectation and orientation logic formed by his experience and recent perceptions. This means that the road has to be constructed accordingly, so as to be understood, perceptible and recognizable.
Our research was aimed at finding out, how, in the dynamics of driving in intersection crossed by the tram, do drivers perceive and identify road situations, change in them, and the regulating actions they imply. We consider the causes of driver's operational error as the first step in a chain of actions, which may proceed to unwanted events. This paper describe our methodology for the analysis of driver behavior in real driving conditions, involving two techniques: the observation and measurement of two intersections crossed by tram at Rabat city, which were selected according to the number of collision and related incidents that occurred there combined with an assessment of the driver's behaviour during a journey made from within the vehicle itself. The merits of such an approach are highlighted and the main results are discussed and interpreted

Keywords: human factor, cognition, intersection, tramway, road environment, representation and perception logic

1 Introduction

The modern tramway has resurfaced as the cure to today's urban transport problems such as pollution, road congestion and uneven access to transit. However trams at intersections often experience frequent and extended delays due to vehicles crossing the tram tracks. There is an increased potential for conflict between trams and vehicles at these locations and crashes are common. Indeed from 2011 to 2013, it was reported a total of 222 tram-involved crashes in Rabat city, 4 fatal, and 81 with serious injury consequences. Of these, 125 (56%) involved tram-to-vehicle collisions at intersection.
Statistical data prove that there is a need to conduct research about the influence of tramway and surrounding environment, (specifically at crossroads) on car drivers' behaviour; the focus remains on creating an environment that minimises likelihood of serious injury.
With this focus in mind, our study aim to allow a systematic sorting out of the different malfunctions which intervene in the genesis of unwanted event at intersections crossed by tram. It is particularly important to recognize that the drivers' behaviour in intersections crossed by the tram is not only influenced by the rules of the tramway priority in the intersection but also by the road features that influence right or wrong behaviour of the driver.

Our analysis is based on the human factors concept taking into consideration the triggers of the driver's reactions and patterns of behaviour, which may result in an accident or even near miss. The systematic study of driver behaviour in real driving conditions makes in our view a useful contribution for such analyses.
The following article first situate the human aspect involved in driving activity, then for the purposes of illustrating our approach, we will briefly discuss some research method and their interest for understanding the way drivers behave in the situations concerned, we present in the last part our experimental study to collect data on driver performance at intersections crossed by tram and the primary finding of our research

2 The driver's ability to understand the road intersections crossed by tram

In order to better understand how we function as drivers at intersections crossed by tram we first turn to Endsley's thoughts on how human operators have evolved during the evolution in interaction with the world. According to this author, situational awareness is the perception of elements in the environment within a volume of time and space, the comprehension of their meaning and projection of their status in the near future,[1]. Applying Endsley's model to the driving environment, the driver firstly perceives road situations. The driver then has to comprehend what they have perceived, by forming "a holistic picture of the environment, comprehending the significance of those objects and events". Finally, the driver takes this understanding, and projects into the near future, in order to take the most appropriate action [2]. Situation awareness plays a vital role in driving as in every dynamic decision making. So driver performance can be evaluated through three parameters: the information received, the interpretation of this information made and the decision taken. An explanation of inappropriate driver behaviour should be sought at each of the different levels of situation awareness.

2.1 Driver's perception of the road

The perception of the road environment, especially its hazards, would be expected to influence driving performance, [3, 4, 5]. There are various level of perception that depends on the stimulus and the task confronting a person. The most basic form of perception is simple detection; that is, determining whether a signal or target is present, [6]. According to the literature [3][7], perception is more than just sensations; it uses memory, classifications, comparisons, and decisions to transform sensory data to a conscious awareness of the environment. This means that through our senses we get an immediate perception of the road environment and what is about to happen around us and we are unconsciously adapting our actions to the road. The physical design of the road and its various parts is the most important source of information to the driver. Drivers perceive the road and the traffic conditions as a whole. In this way, the drivers are the most perceptive of the information they need the most. Indeed drivers seem to be equipped with a well-developed "skip function", they try to acquire messages on aspects of their surroundings that they believe are relevant while ignoring those without useful information. It is an active process that requires significant discipline, as well as knowing what to look for, when to look for it and why.

2.2 Expectation and representation

In the field of cognitive psychology, the notion of representation refers to the idea of internal model developed by the subject for dealing with complex situation. Our understanding is built by combining observations from the real world with knowledge and experience recalled from memory, [8, 9] The driver follows the road with expectation and orientation logic, which was formed by his experience and recent perceptions [10]. Lunenfeld and Alexander [11] define expectancy as "a driver's readiness to respond to situations, events, and information in predictable and successful ways". The expectations are functional for the driver in the sense that they are an important prerequisite for well-adapted driver behaviour [7].

The driver's expectations, however, will be dysfunctional in case he encounters an unexpected traffic situation. When the driver has false expectations of the road and traffic conditions, this mean that he well have an image or idea of what it is to happen which is not in conformity with reality. Indeed the drivers get "confused" and surprised when the road and traffic conditions are not in line with their expectations. In a situation where prompt action is required, problems often occur because there is no enough time to take proper actions, which implies a high risk of missing important information and making wrong decisions, [12].

In order to meet the driver's expectations, roundabouts crossed by tram have to be recognisable, distinguishable, interpretable and safe. This means that they must be designed in a consistent and standardized manner.

2.3 Driver's ability to read the road and self–explaining road

In the road safety context, Salmon and al [13] argue that incompatibilities in situation awareness across road users lie at the root of conflicts between them. A malfunctioning road and traffic environment is identified by the fact that drivers do not behave as intended. A large occurrence of deviant behaviour then indirectly shows that the drivers misunderstood and were surprised by the road and the traffic environment. Because the driver's reaction characteristics cannot be changed, the attention should be focused on a self-explanatory road design.

Concepts such as "positive guidance"[11], "road readability" and "self explaining road" [14] all these terms have been applied to those road designs that communicate to the driver what type of roads they are, or that can be easily categorized by drivers as requiring specific kinds of driving behaviour. They seek to identify the relevant infrastructures features likely to provide a clear picture of the functionality of the road space: how to cross an intersection crossed by tram, who has priority, what kind of information can be expected and so on. Designing road environments that facilitate the 'connection' of different road users' situation awareness therefore seems to provide an appropriate way to reduce collisions between them.

These questions raised from this perspective have led us to formulate our general hypothesis of research, which can be defined as follows: Driver behaviour at tram crossroads will depends very much on what is seen or "not seen" by the driver, in the road scene and how he "reads" the situation. Our research aims to evaluate which expectations the drivers have to tram crossroads and how they drive on it.

3 Overview of research method

In order to do analysis on road traffic dealing with human behaviour, it is required a great amount of data. The range of methods typically used includes observations, questionnaire, verbal protocol analysis and experts (police, road engineers, etc). In depth analysis normally requires experimental observations or field observations with specific supporting equipments. The general rules regarding data collection are (a) not to rely on one source of information and (b) to beware of potentially misleading information.

For the purposes of illustrating our approach, we will briefly discuss some research method and their interest for understanding the way drivers behave in the situations concerned.

3.1 Observation

Observation consists of recording behaviour during task performed. The three main ways that observational data are collected are direct observation, indirect observation, and participant observation. These methods can be used in combination with each other in order to obtain a broader understanding of the task. According to Stanton [15], one of the main concerns with observation is the intrusiveness of the observational method. Observing people affects what they do; people observed can bias the results as they might perform an unrepresentative range of tasks; and the way in which the data are recorded could compromise the reliability and validity of the observations. Overcoming these potential problems requires careful preparation and piloting of the observational study. In planning observational studies, a researcher identifies the variables to be measured, the methods to be employed, and the observational time frame. [16]
For our study, the observation was based on an analysis grid that we have developed to carry out the assessement, and to examine whether it would be possible to identify characteristics of intersection crossed by tram that coincide with a higer likelihood of unwanted events. To guide our choice of elements to be included in this grid, we were inspired by the work of Millot [17] taking care to adapt the reading points to our own field of study and to our specific questions, [18].

3.2 Verbal protocol analysis

Verbal protocol analysis (VPA) is a method which involves participants 'thinking aloud' as they perform a task. Verbal protocol analysis uses data from verbal transcripts to analyze the content there in. These transcripts could come from protocols gathered from recordings of live performance of the task. The transcript can be coded and analyzed at various levels of detail, from individual words, to phrases, to sentences, to themes [19]. Verbal protocol analysis has found use within human factors research as a means of gaining insight into the cognitive underpinnings of complex behaviors. Walker [20] has pointed out the importance of Verbal protocol analysis; he stated that" Within the context of exploring hypotheses and conducting studies in naturalistic settings, verbal protocol analysis can be extremely useful and has much to offer".
To take into account all of these characteristics and the construction of driving in connection with a given situation, we felt that it was essential for our study, to put drivers in real driving situations. The methodology used entailed making observation from within the car during driving on two intersections crossed by tram. At these intersections we consider the point of view of drivers on how they carried out the activity, in order to collect explanatory data on it. This technique enables driver to make comments about his behavior, especially as regards to the environmental conditions which influenced him and came into play in the various situations he encountered.

3.3 Interviews

The interview is one of the original methods for gathering general information, and it has been popularly applied across a range of fields. Interviews can be classified into 4 types of informal conversation, semi-structured, standard structured and focus group interviews, [21]. Greenberg [22] proffers the idea that the task analyst can discover what tasks people perform and develop task descriptions through observation and then validate the descriptions through interviews with the same people. Annett [23] argues that data collection should comprise observation and interviewing at the very least.
Based on the literature review above, a specially designed questionnaire was carried out for the purposes of our study. The questionnaire was composed of three parts. The first part of questionnaire concerns the driver characteristics (gender, age, level of education), the second part concerns information about the driving characteristics of the interviewee (driving

experience, yearly kilometers driven, etc) and the third part of questionnaire contains a basic questions on the road design, the meaning of road signs related to tram, behaviour of other road users, and general questions about the problems that drivers could be experiencing with the tramway crossroad.

4 Study methodology and preliminary findings

4.1 Validation

As a first step, a validation exercise was carried out to assess the feasibility of the methodology. Two crossroads (see figure 1 and figure 2) at Rabat city were selected according to the number of collision and related incidents that occurred there in the last three years from 2011 to 2013.
During the validation exercise, observations were carried out. This involved spending periods of time at each crossroads and observing the behaviour of users. A review of the physical aspects of each crossroad and its surrounding were made. An assessment of the driver's behaviour during a journey within the vehicle itself was also used. A detailed explanation of the driving process was provided by the drivers as they progressed through the sequence of each roundabout, after the journey, the participants were asked to answer a questionnaire with relevant questions about the scene in order to better determine their knowledge and their representations of the driving situations studied.

4.2 Preliminary findings

The following list highlights a number of human factors and design features at each crossroads that can have an influential role in unwanted events and contribute to unsafe conditions. The list can be supplemented by further observations and interviews:

(i) At crossroad Al MANSOUR place at Rabat

Figure 1 Tramway crossroad AL MANSOUR PLACE

- A high traffic volumes and pedestrians movement due to shops, this requires car drivers to be alert for pedestrians and tram approaching;
- A stop line is not demarcated, either with a white line or appropriate pavement marking, to attract the driver's attention and help drivers identify safe stopping location;
- Tramway signs are not always conspicuous due to their position relative to other signs;

- Many drivers indicated that unfamiliarity with the tramway crossroad or driving at night would make the maneuver more challenging;
- Parked cars before and after the tramway line may result in vehicle drivers slowing and being caught on the tracks while a tram approaches
- Non compliance with traffic signal encouraged sometimes by police due to problems of traffic congestion;
- The sequence of lights is believed to be confusing in whether the lights are functional or not; the waiting time is perceived to be too long for all car drivers interviewed, which can be considered as an agent for psychological stress.
- The crossing being completed without the necessary caution increases during rush hours and on working days;
- Text in association with trams signs can be hard to read due to car driver's speed;
- For many drivers there is no difference between all the roads signs related to tram and they just ignore these roads signs so they cannot understand what is expected for them to do.

(ii) At crossroad street IBN ROCHD

Figure 2 Tramway Crossroad Street IBN Rochd.

- At some locations there is considerable visual clutter from surrounding infrastructure, which can detract from the primary safety messages "give way to tramway" and "beware of tram";
- Text in association with trams signs can be hard to read due to car driver's speed;
- Some users failed to fully understand the meaning for various trams signs and aren't aware of the rules and procedures for correctly using trams crossroads;
- Drivers can be distracted from scanning for trams while they seek an appropriate gap in traffic on main road;
- A stop line is not demarcated, either with a white line or appropriate pavement marking to help drivers identify safe stopping location;
- Vehicles waiting or queuing across the tram tracks due to traffic congestion;
- The curvature of the intersection creates a difficult angle from which to observe on-coming trams;
- The desire to pick gaps generated by traffic platoons can result in crossing being completed without the necessary caution, indeed the amount of time the users expect to wait at trams crossroads may influence their risk taking behaviour;

5 Conclusion

The conclusions of the preliminary findings of our study has identified a number of problems associated with tramway crossroad and has explored some of the road design and human factors that contribute to the difficulties experienced by drivers at these intersections. Evidence has been found that at the intersections visited , road scene elements and traffic condition play a role in the difficulties that drivers encounter when crossig these intersections. It is clear that much further investigation is required on the causal factors of errors and on the implications that these driver errors have on tram crossroad safety.
The second stage of the study is now to address the relationship between the elements of the tram environment and types of driver errors; this work is on progress, with the aim to provide suggestions for minimizing potential conflict between cars and trams and for enhancing error tolerance at tramway crossroads.

References

[1] Endsley, M. & Kaber, D.: Team Situation Awareness for Process Control Safety and Performance, Process Safety Progress, Vol. 17, No. 1, 1998.

[2] Bailly, B.: Conscience de la situation des conducteurs : aspects fondamentaux, méthodes et application pour la formation des conducteurs, Thèse de doctorat, spécialité psychologie cognitive, Université LUMIERE Lyon 2, 2004.

[3] Dewar, R.: The Role of Human Factors in Road Safety Audits, Proceedings of the 18th Canadian Multidisciplinary Road Safety Conference, Whistler, British Columbia, June 8-11, 2008.

[4] Gugerty, L.: Situation awareness in driving, Handbook for driving simulation in engineering, medicine and psychology. Ed J.Lee, D.Fisher. CRC press

[5] Grane, C.: Haptic perception, attention and effects on performance in driving context. Doctoral thesis. Lulea university of technology, 2012.

[6] Sanders, M. & Mc Cornick, E.: Human factors in engineering and design, Seventh edition, Mc Graw-Hill, Inc.

[7] Helmers, G.: Explanatory model for road user behaviour. Implications for the design of road and traffic environment, Nordic human factors guideline, January 2014.

[8] Saad, F.: In depth analysis of interactions between drivers and the road environment, 4the workshop of international cooperation on theories and concepts in traffic safety (ICTCT), Vienna, pp. 65-79, 1991.

[9] Norman, D.: some observations on mental models, In: Gentner D., Stevens, A., Mental Models, Hillsdale, New Jersey, London, 1983.

[10] Human factors guidelines for safer road infrastructure, www.piarc.org, 01.01.2012.

[11] Lunenfeld, H. & Alexander, G.:Human Factors in Highway Design andOperations, Journal of Transportation Engineerin g, pp. 149-158, 1984.

[12] Weller, G. & Dietz, M.: Definition, comparison and evaluation of existing self-explaining road approaches in Europe, Deliverable Nº 1, SWOV, June 2010.

[13] Salmon, P. M., Young, K. L., Cornelissen, M.: Compatible cognition amongst road users: the compatibility of driver, motorcyclist, and cyclist situation awareness. Safety Science, 56, pp. 6-17, 2013.

[14] Saad, F.: In depth analysis of interactions between drivers and the road environment. 4th workshop of international cooperation on theories and concepts in traffic safety (ICTCT), Vienna, pp. 65-79, 1991.

[15] Stanton, N.: The psychology of task analysis today ,the handbook of task analysis for human-computer interaction, edited by diane Diaper and Neville Stanton

[16] Wickens C., Lee, J., Liu, Y. & Becker, S.: An introduction to human factor engineering, second edition, Person prentice Hall, 2004.

[17] Millot, M.: Etude des liens complexes entre forme urbaine et insécurité routière, La collection rapports technique du CERTU, 2004.

[18] Moutchou, F., Cherkaoui, A. & El Koursi, E.: Integrating human factors in the analysis of the interaction Tram-Car Drivers,2nd International conference on road and rail infrastructure – CETRA 2012, 7–9 May 2012, Dubrovnik, Croatia, pp. 1093-1099.

[19] Stanton, N.: Behavioral and cognitive methods ,the handbook of task analysis for human-computer interaction, edited by Diane Diaper and Neville Stanton

[20] Walker, G.: Protocol verbal analysis ,the handbook of task analysis for human-computer interaction, edited by Diane Diaper and Neville Stanton

[21] Le Thu Huyen: Risk analysis, driver behaviour and traffic safety at intersections in Motocycle-Dominated traffic flow, tuprints.ulb.tu-darmstadt, 2009.

[22] Greenberg, S.: Working through task- centered system design, the handbook of task analysis for human-computer interaction, edited by diane Diaper and Neville Stanton

[23] Annett, J.: Hierarchical task analysis, the handbook of task analysis for human-computer interaction, edited by Diane Diaper and Neville Stanton.

IMPROVING THE RESILIENCE OF THE METRO VEHICLE TO BLAST AND FIRE

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Abstract

In the recent years, the occurrences of terrorist attacks in metro systems have increased noticeably, as illustrated for instance by the Tokyo, Paris, Madrid, and London attacks. Indeed, several efforts aim at improving the resilience of the metro coaches in order to Increase the resilience of metro vehicle to terrorist bomb blast, through the selection of vehicle materials and structural design, to speed-up the recovery following an attack, allowing the rail system to return to normal operation quickly and to reduce the attractiveness of metro systems as a target for attack. Such efforts include the European-funded FP7 SECUREMETRO project, which reached its conclusion in June 2013, and gathered 11 partners from United Kingdom, Spain, France and Italy. SECUREMETRO aimed at bringing new solutions in fighting the consequences of damages caused by internal blast in metro vehicles. A first full-size blast test was performed to assess the performance of current rolling stock faced to a blast. The second trial consisted of a series of small-scale blast tests of unitary material samples in order to assess their behaviour and the improvement over existing materials. The final stage of the project then consisted of integrating the new solutions in a demonstrator vehicle, and submitting it to the same blast test as the existing vehicle. The paper presents the results of the SecureMetro projects to improve the resilience of the metro vehicle to protect passengers and staff.

Keywords: metro vehicles, terrorist attacks, blast test, blast-resilient certification

1 Introduction

Considerable effort is being devoted by researchers and stakeholders in order to improve the safety of metro systems with regard to terrorist attacks. The issue has been addressed by many researchers, taking into account the bow-tie model of safety management, in which the node is the terrorist attack, with on one side the causes of the attack (e.g. political issues, technical weaknesses) and on the other side the consequences (e.g. human, organisational, economical) [1], [2].

The common goal is to implement a line of defence to isolate these causes and consequences, in order to prevent the attack from occurring and, should it happen, to mitigate its consequences as much as possible. Several research projects have addressed specific elements, or full sets of integrated solutions implementing technological and organisational measures to increase the effectiveness of this line of defence. An example of such current project is SECUR-ED (http://www.secur-ed.eu) which aims at providing and demonstrating a full, interoperable set of tools. Another example is PROTECTRAIL (http://protectrail.eu) which aims at designing a scalable solution integrating a modular set of sub-mission protection tools for railway security, such as passenger clearance control, electrical or communication systems. A last example is MODSAFE (http://www.modsafe.eu), focused on the establishment of a common European strategy including safety and security measures.

SECUREMETRO [1] aimed at bringing solutions to mitigate the consequences of internal blast in metro vehicles. It adds up to more conventional approaches to reduce the casualties and disruptions related to terrorist attacks, although its findings are also relevant to accidental events.

2 Findings from the analysis of existing attacks

SECUREMETRO concentrated on specific cases chosen for their representativeness and the abundance and relevance of the documentation. These cases are notably the bomb attacks in London (2005) and Madrid (2004), and fires in the Daegu metro (2003) and the Kaprun funicular (2001). Other related situations, such as 9/11 or fires in high buildings and other places from which evacuation is difficult, were also considered. The project found considerable insight in the previous work carried out in these fields. As the databases include all the types of terrorist attacks perpetrated during the period, we disaggregated data according to the aim of the SECUREMETRO project. Considering the target of the attack as the main filter for the selection of relevant cases, we focused on events involving strictly rail-based public transportation assets. The tactic used for perpetration is of particular importance to devise ways to mitigate the effects of the attack. Out of the 833 attacks in the data bases, bombing is used in 73% of the cases. If we consider the weapons [Table 1], bombs are by far the most used way to carry out attacks, not only bombings per se, but also other types of attacks such as sabotage, threat or mixed tactics. These proportions have not significantly evolved over the last decade.
This appears even more clearly in the number of victims [3], [4], with 70% of the fatalities (2,541 out of 3,457) and 77% of the injuries (7,832 out of 10,682, if we don't take into consideration the 5 killed and 5,205 people injured in the sarin gas attack against the Tokyo metro in 1995, which remains the only one of its kind so far) caused by bombing during the considered period, making it the deadliest type of attack.

Table 1 Weapons used to carry out attacks on rail in the last 50 years

Weapon used	Percentage of all attacks, 1960-2010 period
Bomb	73%
Firearm	6%
Firebomb-Molotov	4%
Fire	3%
Landmine	2%
Grenade	1%
Material obstacle	1%
CRBN material	1%
Rocket	1%
Knife	0%
Car-bomb	0%
Other	1%
Unknown	7%

An explosion causes damage through several mechanisms, causing specific types of disruptions and calling for different mitigation measures. The primary effects are the shockwave and the blast, as well as primary fragments from the bomb itself: pieces of its case and artefacts such as nails (Figure 1). These effects are inherent to the bomb and can only be alleviated by shielding or dampening measures to dissipate the blast and to prevent the propagation of fragments. Another effect is the creation of secondary fragments, caused by fragmentation of the inner structures, walls or windows, turned into shrapnel. The last mechanical effect is the loss of structural integrity, causing collapse of overhead equipment or of the roof. Although it

is impossible to design a 100% infallible protection system, it is possible to greatly improve the protection of the passengers from falling debris which is a major cause of head injuries. It was found that non-incendiary bombs do not usually cause a fire, and that the current EN45545 standard is adequate to prevent fire in these conditions.
Although the inner structure of the vehicle was mostly destroyed in the studied cases, the overall structure, wheels, bogies and floor, was mostly intact, allowing access and even towing away. This observation, that holds true even with high explosive loads, motivated us to consider resilience, both from the technical and organisational points of view: if a bombed vehicle has surviving passengers and is still able to operate, even in a very limited manner, it is desirable to add resilience in the design to ease and speed up the rescue and the recovery of the system after the blast.

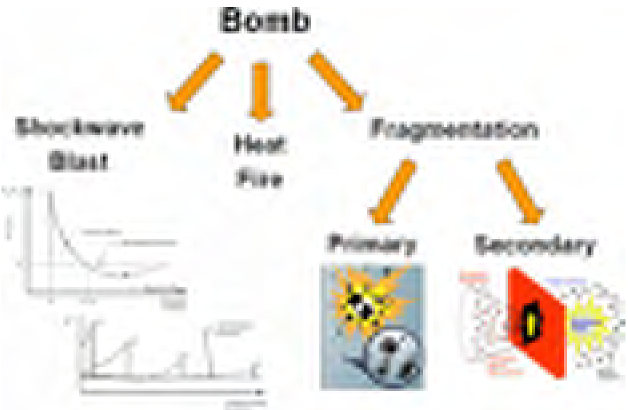

Figure 1 Effects caused by the detonation of a bomb

3 Designs improvements

Particular attention was paid to specific key systems and components of the metro vehicle that must be considered for the safety of the passengers on a bombed train:

- Windows/glazing: should be able to deform plastically, absorbing part of the energy of the blast and minimizing the fragments. High resilience, high plasticity materials should be employed to avoid glass shards. Appropriately supported laminated glass panels have far superior properties that allow them to absorb energy with little or no fragmentation. Another key is the support system: particular attention was paid to the connection between the window and the frame, and between the frame and the carriage structure.
- Walls should be able to deform plastically without fragmenting or breaking into large pieces that could be projected inside by the negative pressure following the blast wave.
- External doors: current rolling stock metro trains have sliding doors, whose mechanism can be severely affected by blast. External doors should not be thrown away.
- Interlocking doors: results from the RAILPROTECT project show that internal walls can contain blast, limiting its effect inside the carriage. Modern trains without interlocking doors can be improved using transparent transversal walls.
- Roof breaks into large pieces under the effect of the blast wave. The roof panels should not fall inside the carriage and injure the passengers. Mechanical systems, like tethered cables, should prevent the panels to be thrown inside.
- Seats: the most important issue is their connection with the vehicle structure, because the blast wave could tear them away from the floor.

- HVAC and other heavy equipment are typically located on the roof of the carriage. As a result they can fall inside the vehicle and hurt the passengers. They would be best placed under the floor, protected by the strong bogies structures.
- Critical systems: since the floor is little affected except at the location of the explosion, all the main supply and critical systems may be protected by placing wires, pipes, etc., inside steel/aluminium cases and ducts placed beneath the floor. A survivable driver's cab could allow driving the train to the closest station, which would dramatically ease rescue and evacuation. All this would increase the weight of the train, so attention should be paid to the mass of the carriage components and to the materials used.

The modifications required to make a metro train blast safe could require a very high investment, which not all could afford. A possible solution is a sort of "blast-resilient certification" for metro lines. Cities where the risk of metro bombings is low may not be interested in a large investment for blast resistant trains. On the contrary medium/high risk cities like London, Paris and Madrid could be interested in being certified as "blast resilient".

4 Experimental assessment

4.1 Test on existing carriage- Metro de Madrid

A full-size blast test was performed to assess the performance of current rolling stock. This test was carried out on a decommissioned Series 5000 carriage given to the SecureMetro project by Metro de Madrid (MdM) [5]. This type of carriage is typical of the rolling stock built in the mid-seventies and still in operation today (Figure 2). The trial consisted of detonating a charge similar to those used in the London attacks, located at the centreline of the carriage. The two doors on both sides of the carriage, at the explosion point became detached and landed 60 meters away. Several other passenger doors were removed from the carriage, while others were buckled and inoperable. All the windows on the right side within the passenger area were shattered, their rubber seals dislodged from their frames.

Figure 2 High-speed image of the trial with an existing MdM carriage

In the driver's cab, all the windows and doors were dislodged, except the access door which remained operable. Damage to the chassis, however, was limited to the rear heat exchanger pack being broken from its mounts, while the central pack was only broken at its rear point. The air reservoir remained intact. Overall, the chassis fared well and the carriage could be towed away, confirming that improved resilience in the passenger area would improve dramatically the security in case of blast. Analysis of the effects, based on this test and the case studies, identified several key areas for improvement, mainly:

· Windows
· Doors
· Interlocking doors
· Components materials
· Joining techniques
· Information and communication systems
· Evacuation system.

Computer simulation was performed to better understand the behaviour of the shock wave inside the carriage, identify the most stressed parts of the internal structure, devise and select structural improvements to improve resilience. This led to a series of small-scale blast tests of unitary samples to assess the improvement over existing materials.

4.2 Test on new demonstrator

The final step then consisted of integrating these solutions in a demonstrator vehicle [Figure 3], and submitting it to the same blast test as the existing vehicle. The improvements brought to the demonstrator consisted notably of:
· Improved resistance of the windows that cleanly separate from the body and do not shatter, thanks to protective film and bonding.
· Improved resistance of the ceiling panels and ceiling-mounted elements using retaining cables to the main vehicle structure: the ceiling does not fall on the passengers, and does not cover the ground with debris which would make walking difficult and hazardous.
· Reinforced lighting using LEDs.
· Reinforced driver's bulkhead.
· Use of flexible backing layer (polyurea) on key elements of the secondary structure to improve flexibility and avoid fragmentation.

In the blast test the driver's bulkhead failed, which the simulation had not predicted. However, the pressure data close to the bulkhead allowed to determine the reasons of the failure, and to propose the necessary reinforcement. The floor was punctured below blast point but did not cause structural failure. Several window bonding solutions were simultaneously tried, and none shattered except the unprotected (reference) window which failed as expected. All the bonded windows stayed in place. The emergency window, protected by film but not bonded to the structure, was ejected in one piece: leaving the frame open but free from glass fragments, allowing safe egress.

Figure 3 High-speed image of the trial with the demonstrator carriage

The seating remained mostly intact, except the seats closest to the charge which were fractured but did not generate secondary fragments. The ceiling panels attached by cables to the main vehicle structure remained attached and did not fall, making walking in the carriage much easier and safer than in the unprotected carriage. Panels attached to secondary elements fell to the floor, showing the importance of tying to the main vehicle structure. The prototype LED lighting worked throughout and after the trial. Overall, these results confirmed the improvement to the resilience of the metro vehicle, passengers and staff. Another achievement of SECUREMETRO is the design of a testing set-up that was an improvement over that used during the first trial.

5 Conclusion

The outcome of the project is expected to yield improvements in the design of metro vehicles to improve blast resilience, and their testing led to the design of measurement methodologies suitable for the specific context of an explosion inside a carriage. It therefore appears that the raised issues, and the solutions found, are both worthy of consideration by the standardization bodies. In addition, adopting low-cost solutions into the design and manufacturing stages will not lead to a marked increase in the cost of rolling stock. For retrofit and future construction, it opens up a new competition in the rail industry supply chain to provide blast-resilient solutions. The results of the work is expected to led to improvements in the design of metro vehicles in the direction of improving their blast resilience, and their testing led to the design of measurement methodologies suitable for the specific context of an explosion inside a carriage. It therefore appears that the issues raised, and the solutions found to these issues in this specific context, are both worthy of consideration by the standardization bodies and insufficiently taken into account by the standards available today, including those pertaining to the related issues of crash worthiness, that take into account some relevant aspects but miss important points related to the pressure wave and its structural effects for instance on doors and windows, and on the environment of the train.

Acknowledgements

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References

[1] SECUREMETRO web site: http://securemetro.inrets.fr

[2] Khoudour, L., El-Koursi, E.M., Velastin, S.A., Buch, N., Lim-Thiebot, S., Fontaine, F.: "An approach for protecting a critical transport infrastructure", Journal of Rail and Rapid Transit, Part F, Sage Publications, pp. 1-11, 2011.

[3] RAND Corporation database of terrorist attacks: http://www.rand.org/nsrd/projects/terrorism-incidents.html (accessed 7/9/2013).

[4] University of Maryland Global Terrorism database: http://www.start.umd.edu/gtd/ (accessed 7/9/2013

[5] Bruyelle, J.-L., O'Neill, C., El-Koursi, E.-M., Hamelin, F., Sartori, N.: Improving the resilience of metro vehicle and passengers for an effective emergency response to terrorist attacks" (Safety Science Journal, Safety Science, n° 62 (2014), pp. 37–45, http://www.sciencedirect.com/science/article/pii/S0925753513001720

THE IMPLEMENTATION OF INTELLIGENT INFORMATION SYSTEMS TO INCREASE SAFETY IN RAIL LEVEL CROSSINGS

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Abstract

On the HŽ-Infrastructure network there are total 1503 level crossings (LC). LC are the intersections of rail and road traffic in the same level. As such they represent a point of high risk on which often comes to extraordinary events with the worst consequences. The most common accidents are caused by motor vehicles drivers (95%), who are unfortunately the most endangered in railroad accidents. Raising safety awareness to a new level is carried out with technical and technological solutions and actions that encourage traffic discipline, drivers and pedestrian culture when crossing the rail tracks. To improve traffic information provided to vehicle drivers, it's necessary to implement intelligent information system (ITS-Intelligent Information Services). Intelligent Information Services is the provision of timely and effective information of dangerous situations (real-time warning systems). In this work is presented application of the level crossing geo-referenced data, on a HŽ Infrastructure railway network, to ensure accurate and timely information delivery to drivers arriving at the level crossing.

Keywords: level-crossing, geo-referenced data, improve safety, HŽ-Infrastructure

1 Introduction

Level crossings (LC) are the intersections of rail and road traffic in the same level. Therefore they represent a point of high risk in which often comes to accidents with the worst consequences. Participants of road traffic often suffer from accidents on LC, although in accidents of heavy road vehicles and rail passenger trains can also cause serious injury to passengers and railway workers, and major damage to vehicles in railway transport. Number of killed and injured people, significant material damage due to vehicles damages (road and rail), and significant losses (economic and time) because of the traffic stop - indicate the seriousness of the problems of security and regularity of traffic on the LC.

Table 1

Year	2007.	2008.	2009.	2010.	2011.	2012.	2013.
Number of killed	12	9	15	7	15	8	11

Table 2

Year	2007.	2008.	2009.	2010.	2011.	2012.	2013.
Seriously injured	12	15	17	10	8	15	12

Raising safety at level crossings is implemented with technical and technological solutions and campaign that encourage traffic discipline and drivers or pedestrian habits when crossing the tracks. Although in crashes of train and road vehicles the most vulnerable are precisely drivers of road vehicles, astonishingly, they are the ones who blame for 98% of accidents at level crossing. An additional astonishing fact is that many semi-barriers, which enable the highest level of security, are damaged. Although only 23%, of total number of level crossings on HŽ-Infrastructure network, are secured in this way. Comparable data of accidents on the LC in Croatia, Slovenia, Austria, Germany, France and Denmark are presented in Table 4.

Table 3

Year	2007.	2008.	2009.	2010.	2011.	2012.	2013.
Broken semi-barriers	682	670	706	613	572	534	518

Table 4 The number of Level-crossing accidents relative to train km

Year **Country**	**2011.**	**2012.**
Croatia	0,70	0,69
Slovenia	0,29	0,40
Austria	0,28	0,24
Germany	0,05	0,07
France	0,08	0,07
Denmark	0,03	0,08
* according to – ERAIL (European Railway Accident Information Links)		

It is therefore necessary, in addition to technical solutions and campaign that promote drivers culture and the respect of traffic signs, improving system of informing drivers of road transport by establishing smart information systems.

2 Program and campaign for solving LC problems

2.1 Program for solving LC in RC

Level crossings or rail-road crossings, in security terms are the critical point of the rail and road transport network, and so the quality of solutions should definitely relate to both branches of traffic. They are the main entities that are responsible for the implementation of this program together with the local authorities interested in the general security of the local population.
For all these reasons the Department of railway traffic of Ministry of Sea, Transport and Infrastructure has launched an initiative to create the measures and activities related to systematic and continuous solution to the traffic safety on the LC, which is in cooperation with the working group from HZ-infrastructure resulting in the document entitled ''Program of solving LC in the Republic of Croatia'' (Program).
The Program was conceived as a national program, which is one of the preconditions for a systematic problem solving of LC in big traffic system (HŽ, HC, ŽUC), all administrative, organizational and technological levels of local communities. This Program makes up one segment of the entire "National rail infrastructure program".
The Program has processed the range of technical and technological solutions on the LC; ensuring safety with light/audible ringing signals and half barriers, the removal of crossings with or without reduced roads construction and two-level constructions.

The Team for solving the level crossing issues in HŽ-Infrastructure during the last years, 2007-2013.g., achieved significant results with technical solutions foreseen in the Program:
· 105 – ensuring safety with light/audible ringing signals and half barriers;
· 64 - removal of crossings with or without reduced roads construction;
· 8 - two-level constructions.

In the upcoming period enhanced investments are going to be in this area. Making of a new 5-year 'Program for solving LC' and better cooperation with local governments, as well as Croatian Roads and County Road Administration - will contribute to the acceleration of solving level crossing issues according to the Program.

2.2 Campaign ''Train is always faster''

Educational and promotional campaign 'Train is always faster' HŽ-Infrastructure carried out since 2000 year, in collaboration with Police Departments and primary schools across the Croatian. The target groups of this campaign are; the drivers of road vehicles, school-age children, the local community and media.
Statistics shows that with the increase in the level of security at level crossings does not decrease number of accidents in the same proportion. The general trends are educational-marketing activities aimed at raising awareness of traffic participants. Also, since 2011 HŽ-Infrastructure has actively participated in the campaign marking the International Level Crossing Awareness Day (ILCAD). The campaign synchronizes International Union of Railways in collaboration with the European Commission, in order to change misperception that level crossings accidents are a problem only for railways companies. Program and campaign have achieved many positive effects, and maintain a state without a significant increase in accidents at level crossings in a time of increasing the number of road vehicles and a hectic pace of life.

3 Proposal for increasing security

Existing methods of warning drivers about approaching LC are only with the road signs of approaching the rail, in the 240, 160 and 80 meters before the rail (Figure 1 and Figure 2).

Figure 1 Approaching the level crossing with barriers or semi-barriers.

Figure 2 Approaching the level crossing without barriers or semi-b.

Those signs according to the 'Regulations on traffic signs, signals and equipment on the roads' belong to the group of 'signs for dangers', which stipulates that participants in road traffic approaching to a place where they could be in danger.
Because of the astonishing fact in which motor vehicle drivers are blamed for 98% LC accidents, it is obvious that we need better solutions in alerting road drivers.
The European Parliament 2010th is adopted the 'Directive 2010/40/EU' - on the implementation of intelligent transport systems in the field of road transport and interfaces with other types

of traffic. The goal is improving system of informing drivers of road transport by establishing smart information systems (ITS- Intelligent Information Services).
It is a time efficient, informative and effective warning about possible dangerous situations (real-time warning systems). The information system must be interoperable, applicable and available to as many drivers as possible. Smart information systems are part of the everyday life of the majority of EU citizens. Accurate and timely information about dangerous situations in traffic will help us to increase the level of traffic safety but also to increase the efficiency of the transport system.
ITS which would inform road drivers of approaching the level crossing can be carried out by using geo-referenced data of level crossing on the HŽ-Infrastructure network.

4 Technical solutions for increasing security on LC

4.1 Data collection and processing

Existing data about LC in HŽI has conducted in tables, and for database is used Microsoft Excel. It is good enough for all current and planned future needs and is due to the simple architecture and data using, selected for further use.
Each LC is assigned my unique identifier in the form of a new column ID. In the database terminology, the primary key of a relational table uniquely identifies each record in the table. In this paper we have used a data of the level crossings on wider area of Zagreb.
Absolute positioning via GNSS devices with the possibility of receiving corrections of EGNOS system is chosen for data collection method. During storage in memory, handheld computer, with the number of points and coordinates, each recorded point is assigned an attribute name ID of a level crossing according to the database.
Geo-referencing is aligning geographic data to a known coordinate system so it can be viewed, queried, and analyzed with other geographic data. The collected data are transmitted to a computer with the help of software solutions Trimble GPS Pathfinder Office, and the result is a text file (CSV) with a list of points, the corresponding coordinates (E, N) and a unique ID of each LC. The road navigation and web applications (Google Maps / Earth, OpenStreetMap) use the World Geodetic System (WGS84) datum as the underlying coordinate system for navigation purpose and imagery base (spatial coordinate's ϕ and λ).
To transform rectangular coordinates in Croatian coordinate system to WGS84, official program of Republic of Croatia State Geodetic Administration, T7D for coordinate transformation (Figure 3). For simplicity's sake and difference between HTRS96 and WGS84 coordinates (which is irrelevant for this work), HTRS96 is used.

Figure 3 The official program of the SGA to coordinate transformation - T7D

Coordinate transformation converts data to an ASCII file containing the aforementioned geographic coordinates in hexadecimal system (decimal degrees). The prepared data are ready for use in navigation devices.

4.2 Application for inform drivers of road vehicles

Orientation and navigation has never been easier than today, using GPS technology. Whether for the protection of life, reaching destination faster, entertainment or any other use you can imagine. GPS navigation is becoming an increasingly important in everyday use.
Points of interest (POI) are points stored in GPS device memory called “waypoints”. House, Airport, Level Crossing or famous cultural - historical places that you would like to re-visit are some of the examples of positions that can be saved and later found. In GPS-receiver memory can be created points that show places where you were and GPS-receiver becomes a navigational device showing users the way to go.
For the purpose of informing drivers of road vehicles on the arrival of the railroad POI file for Garmin devices was created. POI file, for each location contains a name, a brief description and coordinates. Attached file could be a picture that will be displayed on the map at locations in the POI file. Sound warning for vehicle drivers is designed out of the soundtrack to alert drivers to the presence of LC (a combination of audible warning light-sound devices and quotations from campaign “Train is always faster”).
Using the POI Loader (Garmin’s free program to record the waypoints in the device) and prepared files, we made POI files with proximity detection (garminZCPZagrebwav.gpi). Proximity detection is set so that you’ll be automatically notified when you’re approaching 100 m of the level crossing. With the same program POI file is transferred to internal memory of a GPS device to be used in road navigation (Figure 4).

Figure 4 An example of the POI file

Short film attached to this paper, shows how road navigations works with the included POI files and audio-visual experience of road drivers when encountering the level crossings.

5 Other possibilities of using geo-referenced data on LC

5.1 Improving LC records

Spatial data obtained this way can be used to connect each LC with DGU Geoportal or display that combines following layers: digital orthophoto maps, spatial units, digital cadastral and others.
When user in Excel “clicks” on a prepared link, default web browser is started with preloaded view centered exactly on the level crossing. From the DGU Geportal for each LC were extracted attribute data on the number of the cadastral parcel and the name of cadastral municipalities (Figure 5).

Figure 5 Web browser view centered on the level crossing

5.2 Use of LC data on the Internet

By collecting data on the field every level crossing was photographed four times;
- a perpendicular to the axis lines, right side of the railroad chainage;
- b in the direction of the tracks, toward the beginning chainage;
- c in the direction of the tracks, toward the end chainage;
- d perpendicular to the axis lines, left side stripes .

Each photograph generic name has been renamed so it contains ID and photographing direction (ID-A, ID_B etc). To view photos online, these were transferred to the Internet using Google services, Picaso Web Albums. The same allows free storage, and makes sharing photos a snap. For each photo automatically generated web address was merged with LC database imported in Google Fusion Tables. So in the browser we have all the data from basic Excel table with pictures for each level crossing (Figure 6).

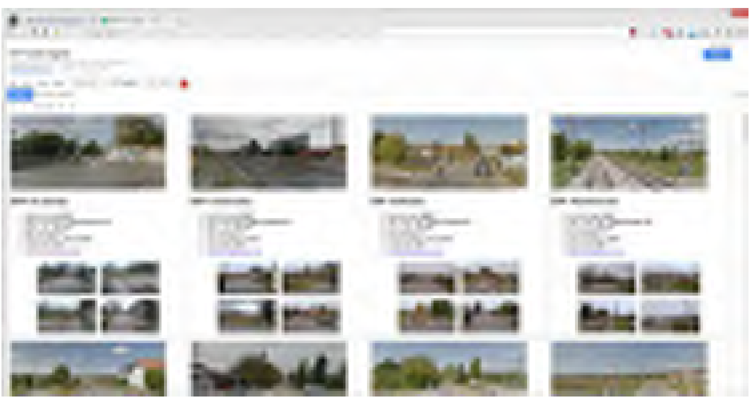

Figure 6 View data of level crossings

By adding view cards in Fusion tables we can visualize LC data on Google Maps (Figure 7). The red dots on a map represent LC's locations. Each dot is interactive, and by clicking on each one you can open an information window with basic information about level crossings.

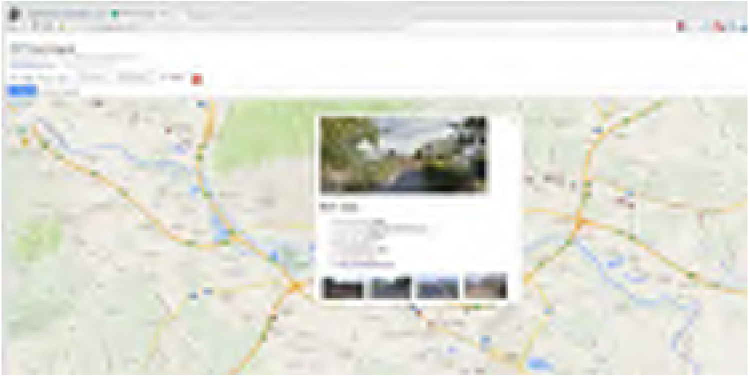

Figure 7 Data Visualization of LC on the map

It is also possible to filter the data so that at any moment user can make queries depending on particular needs, and for all attributes defined in the database. If you wish to display data of pedestrian crossings in the filter, select attribute PP and choose find and Fusion Tables shows the map with filtered data of pedestrian crossings.

6 Conclusion

The level crossings are potentially dangerous point because of its specificity, and the crossing of two types of traffic at the same level. Increasing the safety of movement of vehicles and pedestrians are carried out by technical and technological solutions, and campaign that encourage traffic discipline and drivers or pedestrian habits when crossing the tracks. The number of deaths and injuries, and especially the number of avoided accidents (broken semi-barrier) illustrate the need for additional improvements. By using new technologies it is possible to achieve improvement of the system informing drivers of road vehicles on the dangers when approaching the level crossing. Development of technology simplifies usage of navigation devices in everyday life.
Collecting and processing geo-referenced data on the level crossing network Railways Infrastructure, it is possible to use them as an Intelligent Information Services to inform drivers to approach level crossing. Because of facts that vehicle drivers are the ones to blame for 98% of accidents at level crossing, it can be expected that use of ITS can reduce the number of accidents at level crossings. Collected spatial data can also be used as improvement of information about level crossings, and in every work of Railways Infrastructure employees, external traffic experts, police, local government, etc.

References

[1] Act of safety and operability of the railway system in Croatia

[2] Program of solving LC-a in Croatia

[3] Annual report on security in 2012 - HŽ-Infrastructure

[4] Schütze, E.: Current state of technology and potential of smart map browsing in web browsers, Thesis, University of applied sciences Bremen

[5] Brillhart Dave: Garmin: Custom POI Alerts & TourGuides

[6] Google: Google Maps API

10 ENVIRONMENTAL PROTECTION

WELL-TO-WHEEL ENERGY COMPARISON OF US AND EUROPEAN RAIL FREIGHT

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Abstract

Worldwide, about 30% of the final energy and 62% of final oil is consumed by the transport sector. Reducing global fuel consumptions is one of the highest priorities for all countries for both energy security and greenhouse gas emission implications. With globalization, transport of goods increased significantly and then pressure on the environment. If the departure and arrival of the goods are on the same continent, transportation can be done by air, road or rail. Between this three means of transport, it is usual to consider that rail freight transportation allows reducing energy consumption per ton transported. United States and Europe have developed an important railway system to transport goods and people. These two systems are very different in terms of infrastructure, rolling stock and operation. They are specialized; the American in freight transportation while the European in people transportation. In a first part of this article, these differences are compared and analyzed. Then, a well-to-wheel reasoning is carried out on various types convoy (container or bulk) in Europe and US. Length, mass, propulsion, train resistance, speed, track profile, gauge, electricity generation etc. are taken into account. Train is modeled as a point with a mass. Newton's second law is applied on this point. The total force to the drive wheels provided by the electric motor is computed by a dynamic simulation. Finally, US and Europe rail freight are compared with an energy per ton transported indicator. This energy comparison shows the advantages of the American system on the environment despite the use of diesel locomotive.

Keywords: rail freight, energy, well-to-wheel, Europe, United States

1 Introduction

Globally, transportation activities are using more than 30% of the primary energy consumption, (33% in EU 27, [1]), they cause more than 25% of greenhouse gas emissions [1] and the trend is increasing as shown in Fig. 1.
In a world where energy is becoming more costly, and where global warming is making violent climatic episodes (floods, storms, etc.) more and more common, using environmental friendly freight transportation will soon be a necessity. Freight trains have now, in Europe, a modest market share for long distance transportation, but they are thought as being more efficient both in energy consumption and in greenhouse gas (GHG) emissions than other transportation modes as shown in Fig. 2.

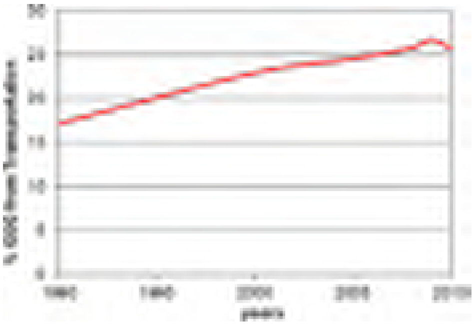

Figure 1 Greenhouse gas from transport in Europe, according to [2]

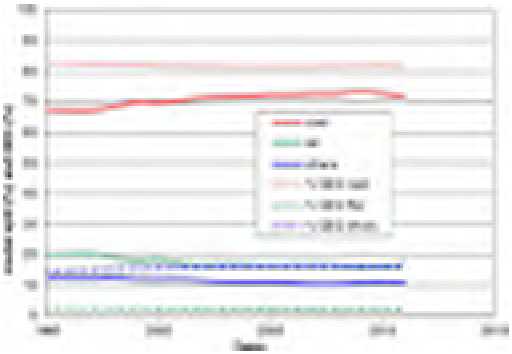

Figure 2 Internal transport modal split in Europe Market share and Greenhouse gases share, according to [1]

2 Typical train

Freight trains come in many varieties and a comparison between typical European and non-European freight trains shows the differences in Tab. 1.

Table 1 European and freight train comparison

	European freight	Non european freight	Impact
Motorisation	1 or 2 Electric engines	Up to 5 Diesel engines	· speed sensitivity to track profile · acceleration and transit time
Couplers	UIC couplers	Automatic couplers	· train length number of engines · aerodynamic resistance
Gauge	GA to GC	Double stack	· competitivity · aerodynamic resistance
Max length	500 to 750 m	2,500 m and more (ECB)	· line capacity competitivity
Speed	100 km/h	60/80 km/h	· transit time · energy consumption
Trafic management	Passenger train priority	Freight train priority	· transit time, number of stops/ starts, energy efficiency

As seen above, the two types of freight trains are quite different, and it is a legitimate question to ask if one is more efficient than the other in primary energy use or in greenhouse gaz effect.

To check what type of train is more efficient in terms of energy consumption or GHG (or both), we tested 4 typical European and US trains:

- An heavy mineral train 87 waggons and 2 engines, 900 m and 10,440 t, named US-t-1 in this paper;
- A double stack container train, 105 waggons and 2 engines, 2,050 m and 10,500 t, named US-t-2;
- An European heavy freight train on a mixed passenger/freight network, 215 m, 1 engine, 1,300 t, noted EU-t-1;
- An european container train, 390 m, 1 engine, 1,300 t, noted EU-t-2.

3 Train resistances

The running resistance has a direct impact on energy consumption of trains. It is important to evaluate it correctly. Hence, in a first step we compare the different formulas proposed in the literature. In a second step, we choose one running resistance formula and we calculate the running resistance for the 4 trains selected in our European/US comparison.

3.1 Formula train resistance comparison

Four train resistance formulas are compared. The fisrt one is proposed by Lukaszewicz [2], data comes from full-scale tests with freight trains in Sweden. The second is proposed by Rochard [3] which summarizes various formulas. The third one comes from Allenbach [4] and the last one is the so called "Davis equation" summarized by Hay [5]. The 4 running resistance formulas are consistent and similar. We found a difference of less than 15% at a speed of 100 km/h for a standard freight train. Subsequently, only the Davis is then presented.
Particular attention should be paid to convert the "Davis equation" into international system units. Indeed, it is necessary to convert the speed in m/s, the mass in kg (US ton is equal to 907 kg) and force in N (1 lbf is equal to 4.45 N). The formula used is express in the equation (1).

$$R_{Davis} = 2.943\times10^{-3}\cdot W_{SI} + 88.96\cdot n + 1.097\times10^{-4}\cdot W_{SI}\cdot V_{ms} + 22.26\cdot K\cdot V_{ms}^{2} \quad (1)$$

where:

R_{Davis} is the resistance in N for one car;
V_{ms} is the speed in m/s;
W_{SI} is the car weight in kg;
n and K are number of axle and drag coefficient.

3.2 Train resistance calculation

Davis equation is used to calculate the running resistance of the 4 trains used in the US/European comparison. The coefficient K used is: 0.07 for US-t-1 and EU-t-1 (conventional equipment), 0.0935 for EU-t-2 and 0.11 for US-t-2 (double stack).
The train resistance for each train is proposed in the Fig. 3. We can observe that the running resistance is largest for US than European trains (Fig. 3, left). This is explained by the fact that the U.S. trains are much longer and heavier. If we compare the running resistance divided by transported mass, it can be seen with Fig. 3, right than the US-t-1 is more efficient. Moreover, we can observe than rock trains (US-t-1 and EU-t-1) are in the both case more efficient than the containers train. Due to high aerodynamic drag, the efficiency of US-t-2 (double stack) greatly decreases with the speed.

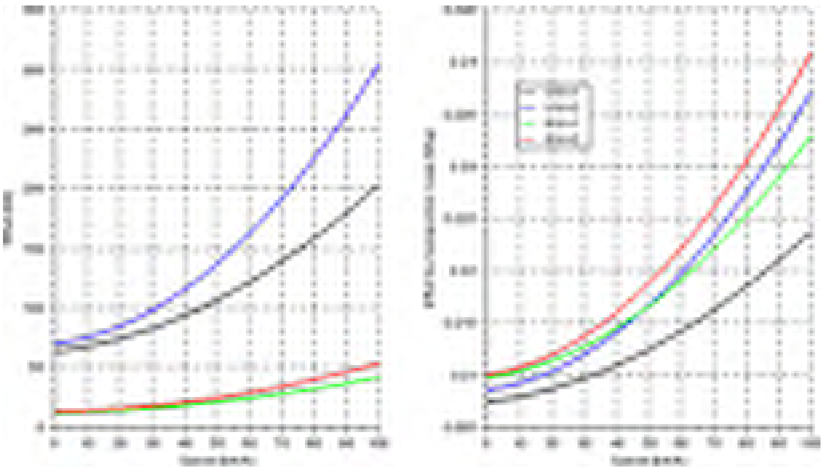

Figure 3 Train resistance (left) and train resistance by ton transported (right)

4 Speed profile simulation

A virtual line of 80 km was created to simulate a representative track and a speed profile. On this path, speed limits are introduced to comply with the regulations and practices in the US and Europe. A dynamic model of a train is used and subsequently presented.

4.1 Track profile

We compare the 4 trains on a track shown in Fig. 4. This 80 km itinerary is characterized by: 3 hill at a gradient of 8, 5 and 4 mm/m and one down at 10 mm/m.

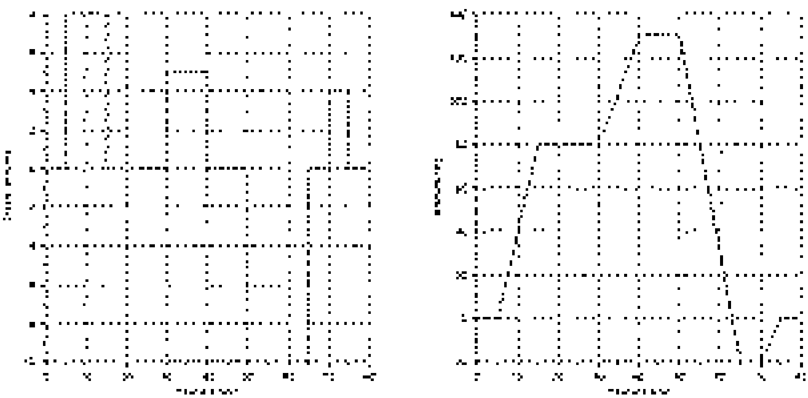

Figure 4 Slope (left) and profile (right) of the line

4.2 Dynamic model

As proposed by Lukaszewicz [2] or Rochard [3], a simplified dynamic model train is used. The train is considered as a point with a mass M. Newton's second law is applied on this point to calculate the total force to the drive wheels (F) provided by the electric motor. The formula is:

$$F = m \cdot k \cdot \gamma - M \cdot g \cdot \sin(\alpha) - F_r \qquad (2)$$

where:

k	conventional coefficient which represent inertia of rotating masses;
m	the mass of the train;
γ	the longitudinal acceleration;
F_r	the resistance force calculated by Davis equation;
g	the gravity acceleration and is local gradient of the line.

The rules of driving are as follows: two limit speeds are introduced. The first one is the set point speed (S_{sp}). If the speed of the train is lower than the set point speed then F is positive. It is the case 1 (traction). The second one is the limit speed (S_l). If the speed of the train is between the set point and the limit speed then F is zero (case 2, coasting). If the speed of the train is above the limit speed then F is negative (case 3, braking).

4.3 Results

We can see in Fig. 5 the simulated speed profiles of the 4 trains. The black line is the speed of the train, the blue dotted line is the set point speed and the red dashed line is the limit speed. The set point speed is 60 km/h for US-t-1, 80 km/h for US-t-2 train and 100km/h for European train. Moreover, to respect the rules of traffic in the country, the set speed is reduced to 25 km/h in the US and 100 km/h in Europe in the slope. To represent a real driving, coasting is introduced for U.S. trains before the speed reduction for slope. It can be seen that the speed profiles are very different between the U.S. and Europe trains. US train speed is more sensitive to the track profile.

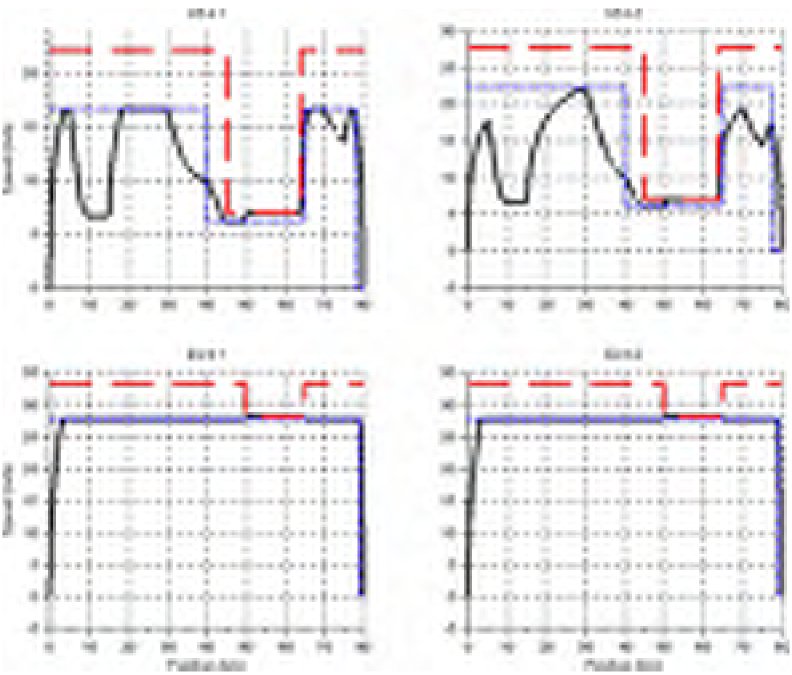

Figure 5 Speed profile of the 4 trains

5 Energy consumption

For all trains, the power transmitted to the wheel P_w is calculated (product of velocity v and force F). Furthermore, a constant P_a is added to take into account the consumption of auxiliary (cooling engine, compressed air system, etc.). Then, the energy consumed by the train E_{train} is calculated using the equation 3:

$$E_{train} = \int \frac{P_w + P_a}{\eta} \, dt \tag{3}$$

where:

η efficiency of the traction system.

For electric traction (European train), η equals 80% (Jeunesse [6] give a ratio of 87% for the French high speed train) and for diesel train (US-t-1 and US-t-2), η equals 45% (according to [7]). Finally, to calculate the primary energy, losses to extract and process oil and to produce electricity are taken into account. The well-to-tank ratio is taken at 1.3 (according to [8]) and the production electricity ratio is taken at 3.1 (according to [8] or [9]).
In addition, the GHG emission indicator is calculated. According to [8], we considered 530 g of GHG rejected for 1 kWh of electricity consumed (emission power generation in Europe) and 92 g of GHG rejected for 1 MJ of diesel burned.

6 Results

The results of our 4 simulations are shown in Tab. 2. European trains consumption is upper than 2 times US trains consumption per transported mass in energy. Moreover, GHG emissions are significantly higher.

Table 2 Primary energy (MJ/tkm) and greenhouse gas emissions (g/tkm) comparison

	US-t-1i	**US-t-2**	**EU-t-1**	**EU-t-2**
MJ/tkm	0.097	0.11	0.25	0.27
g/tkm	7	9	12	13

These results are consistent with data from [9]. According to this reference, the consumption in primary energy is about 0.25 MJ/tkm in Europe. Obviously, the travel time of 4 trains are different. It is approximately 130 minutes for US trains and 50 minutes for European trains.

7 Conclusion

We show in this paper that rail freight is more interesting from an energy and GHG emissions point of view in the United States. Despite the low efficiency of thermal engines, US trains are more efficient for several reasons: technological choices (couplers, electric/diesel traction, etc.) and different operating methods (speed, priority, etc.). In the perspective of this works, other simulations can be performed with different engine efficiency or well-to-tank ratio for instance if more accurate data are available. It would be interesting to work on the Ofoten line in Norway for example. Indeed, this line combines the advantages of two systems: heavy trains with resistant couplers, electric traction and high performance energy mix (electricity production in Norway is mainly generated by hydropower).
We note that the rail freight remains more interesting than road transport from an energy point of view. Indeed, according to [9], road transport consumption is 0,7MJ/tkm (between 3 and 7 times more than rail).

References

[1] European Commission: EU energy in figures. Statistical Pocketbook, pp 116-120, doi:10.2833/15026, 2013

[2] Lukaszewicz, P.: Running resistance – results and analysis of full-scale tests with passenger and freight trains in Sweden, Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit, 221, pp 183-193, 2007.

[3] Rochard, B. & Schmid, F.: A review of methods to measure and calculate train resistances. Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit, 214, pp 185-199. 2000.

[4] Allenbach, J.-P., Chapas, P., Comte, M. & Kaller, R.: Traction électrique. 2008

[5] Hay, W.: Railroad Engineering. SBN-10: 0471364002, 1982

[6] Jeunesse, A. and Rollin, M.: La motorisation du TGV POS. Revue Générale des Chemins de fer, March, pp 19-39. 2004.

[7] Wikipedia contributors: Diesel engine. Wikipedia, The Free Encyclopedia, February 2014

[8] Ecoinvent. Database used: Dones R., Bauer C., Bolliger R., Burger B., Faist Emmenegger M., Frischknecht R., Heck T., Jungbluth N., Röder A., Tuchschmid M. Life Cycle Inventories of Energy Systems: Results for Current Systems in Switzerland and other UCTE Countries. ecoinvent report No. 5. Paul Scherrer Institut Villigen, Swiss Centre for Life Cycle Inventories, Dübendorf, Switzerland, 2007

[9] AFNOR: Fascicule de données énergie et transport, FDP01015, 2006

COMPARATIVE WIND INFLUENCE ON USE PHASE ENERGY CONSUMPTIONS OF ROADS AND RAILWAYS

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Abstract

Sustainable development is largely depending on transportation systems ensuring mobility and safety. This mobility requires a sufficient access to the energy, at an affordable price. According to the peak-oil hypothesis, rarefaction and increasing in price of the non-renewable energies are nevertheless expected.
Even if transportations systems could partly rely on renewable energies, their optimization should involve energy and mobility criteria. In this study, surrounding wind fields and traveling speed are conjointly investigated in order to evaluate achievable reductions in fuel consumptions for both railways and road transportation systems.
In the first part of this work, aerodynamic forces on trains are evaluated with the help of an energy model, validated with experimental tests data. For that, numerical determination of aerodynamic coefficients for several wind and topological situations have been computed with Solid-Works and associated to atmospheric characteristics determined over the considered high-speed line (Rhine-Rhone line; about 140 km) with the AROME model (meteo-france), from measurements at several weather stations.
Simulation results show that for a weak wind and a train traveling at high speed, wind influence is about 5% on aerodynamic forces power. Moreover, for a moderate train speed (about 45 m/s), a slightly stronger wind (about 5 m/s) has an influence of about 30%, compared to the without wind case.
In a second time, road experiments have been conducted with a passenger car equipped with an air flow meter and an oil flow meter, in addition of dynamical measurements as speed, position, altitude... Similarly to the railway case, wind exposition could be considered as a significant road design parameter.
In conclusion, this work based on large sets of full-scale experimental data and the numerical simulation of the aerodynamic forces, points out that wind influence on total aerodynamic power consumption is noticeable, both for railways or road transportation systems.

Keywords: railway, roads, energy, use phase, wind

1 Introduction

Transportation systems as roads and railways are usually designed by considering criteria of time-efficiency, mobility and safety [1,2], and to a lesser extend, energy savings. Negative perspectives on oil availability and climate changes justify the need to enhance the consideration of this last criterion.
Indeed, the production of non-renewable energies as oil, gas and coil is peaking as their demand is growing, this unbalance between oil demand and production being often qualified as the Peak Oil Theory [3, 4]. According to the International Energy Agency New Policies

Scenario [5], world oil production should reach 96 million barrel/day in 2035 on the back of rising output of natural gas and unconventional oil, as crude oil production remaining stable. A large part of the demand growth could come from China alone, driven by rising use of transport fuels [6].
In order to better integrate the energy criterion in the design of road and rail infrastructures, preceding researches on safety have been adapted to the energy evaluation [7, 8]. For example contact forces models have been transposed from safety to energy evaluation [9] and extended to the LCA assessment of railways [10]. Nevertheless, if aerodynamic forces are generally considered as the result of the vehicle speed, with substantial achieved drag reductions [11], surrounding wind field influence studies are often limited to transverse stability [12]. But for modeling the infrastructure-based energy demand, wind field influence remains to be evaluated.
In this paper, the wind influence on energy consumption is evaluated from experimental measurements and for both use phases of a high-speed railway and a road. For that, Ifsttar and RFF have performed full scale tests on a newly built high-speed railway and the wind influence has been later determined numerically from wind field reconstitutions. Wind influence on a passenger car has been verified similarly but with the advantage of having direct measurements of relative wind on the vehicle.
In perspectives, energy models will be helpful for qualifying infrastructure alternatives on energy criterion, for the two predominant transportations modes.

2 Wind influence assessment on train consumptions

2.1 Experimental tests and aerodynamics modeling

The acceptance of work tests of the newly build Rhine-Rhone high-speed line have been an opportunity for verifying the wind influence on train consumptions: electrical consumptions and vehicle speed have been recorded for various speeds and environmental conditions. Wind speed and direction have been afterwards determined over the whole line (140 km long) with the help of the numerical model AROME [13], from measurements at several weather stations (one of the wind field computation in given in Fig.1).
15 runs have been selected for evaluating the wind influence on consumptions, among the 144 runs of the performed tests. This selection managed to keep various combinations of train speeds and wind incidence angles and speeds.

Figure 1 Example of the discretized wind field computed with the AROME model, around the 140 km high-speed line

Aerodynamic coefficients of the train (ALSTOM TGV Duplex) are computed with the help of Navier-Stockes simulations of the air flow on auto-adaptive Cartesian meshes. Particularly, a longitudinal coefficient C_x of 0.148 is found, in accordance with values determined for other studies [14].
The computation of the aerodynamic forces is based on the Eq. (1), which establish the link between the longitudinal aerodynamic force F_x and the relative air flow velocity with the C_x coefficient, the air density (ρ) and the S reference surface ($S=10\ m^2$).

$$F_x = \frac{1}{2}\rho S V^2 C_x \quad (1)$$

Simulations are then done for the 15 selected train runs, for various wind angle directions, wind speeds and train dynamics. At last, aerodynamic power is computed all along the itineraries, while considering situations with or without wind.
Power variations of aerodynamic forces, between simulations cases with or without wind, are given in the Fig. 3 for three tests runs. The green curve labeled HS1 corresponds to a train traveling at a high speed of 95 m/s in a weak mean wind (2 m/s). The blue and red curves, labeled respectively LS1 and LS2, correspond to a train traveling at a moderate, steady speed of 47 m/s for two quite different wind mean speeds (wind speed profiles given in Fig. 2).
As a result, for a weak wind (2 m/s) and a train traveling at high speed (95 m/s), wind influence is yet of +5% and -5% on power due to aerodynamic forces on the train, respectively on sections of the itinerary where wind is forwardly and rearwardly oriented (test run labeled HS1 on Fig. 3). This result is deduced from the analysis of the HS1 test run between the 70 and 120 km abscissa, section for which the mean speed is of 95 m/s.
A more remarkable result is that for a moderate train speed (about 47 m/s), a stronger rear wind (5.5 m/s) lowers the needed power by 30% compared to the case without wind, and a front wind of 3 m/s raises it by 20% (respectively test runs labeled LS1 and LS2; see Fig. 2 and 3, at the considered pk position of 70 km). These results will be integrated in the train/infrastructure energy model, linking railway geometry, environment (wind), vehicle dynamics and energy of use consumptions.

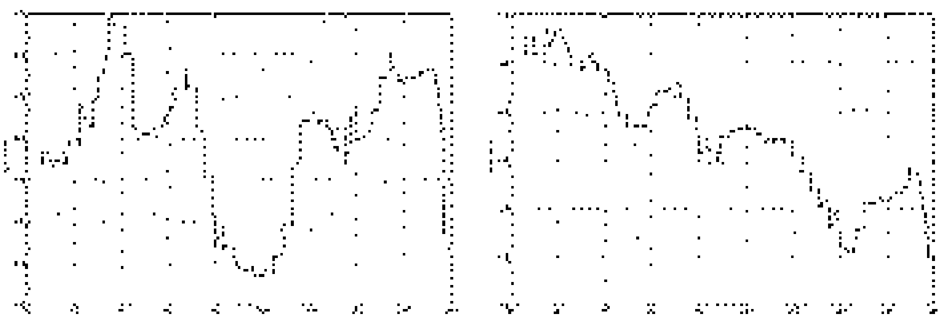

Figure 2 Wind speed for the LS1 (left) and the LS2 (right) test runs along the curvilinear distance pk (km)

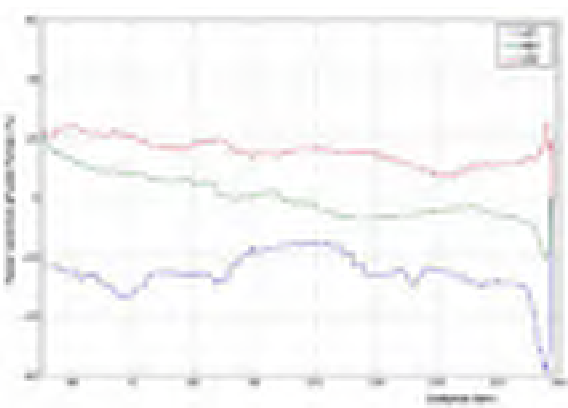

Figure 3 Power variation of wind forces for the LS1, LS2 and HS1 test runs

3 Wind influence assessment on road passenger cars

3.1 Experimental tests

Wind influence on a passenger car has been verified similarly, for various test runs of a middle size gasoline passenger car, a “Renault Clio”. The aim of these tests is, as for the railway case, to enhance a consumption model that could be use for the energy-based comparison of alternative infrastructures.
Tests have been carried out for various sets of wind fields, for intended vehicle speeds of 50, 60, 75, 80, 100, 120 and 125 km/h, for the 4th and the 5th gear ratios, and in the two directions of a 400 meters section of a specific test lane.
All measurements have been recorded with a National Instrument acquisition board at the simultaneous frequency of 100 samples by second. Fuel consumptions are measured with a Kistler DFL transducer for fuel consumption measurement (accuracy ±0.5 % of reading; resolution $330x10^{-3}$ ml; non-contact Hall sensors technology). Vehicle speed is determined with an optical Correvit sensor, air flow speed is measured with two windmill anemometers mounted perpendicularly on the vehicle roof. Other data as Buscan consumption or vehicle speed are recorded too, as a reference basis.
Compared to the railway case, road tests have the advantage of having direct measurements of relative wind on the vehicle, instead of a numerical reconstitution.

3.2 Wind influence assessment

Tests performed for various wind conditions, for several days, confirm a rather large variation of fuel consumption at a given test speed. For example, Fig. 4 exhibits consumptions variations of 3 to 4 l/100km for nevertheless steady test speeds of 48, 73 and 97 km/h (intended test speed of 50, 75 and 100 km/h, variations due to vehicle display shift).
In Fig. 5, these fuel consumptions are plotted in function of the whole corresponding anemometer speed range. As expected, a 2nd order polynomial regression can be established between axial airflow speed and consumptions, since car aerodynamic force is expressed in the same form as given in (1) and that consumption is linked to the aerodynamic force (by engine and drivetrain efficiency, and aside other forces as tire rolling resistance).
For the case of intended vehicle speed of 75 km/h, the wind influence on consumptions is merely pointed out in Fig 6, as consumptions are plotted in function of the A/V ratio of the airflow speed to the vehicle speed. The airflow speed being the addition of the vehicle and wind speed vectors, values of the A/V ratio lower to 1 indicate a rearward wind, and values upper to 1 indicate a frontward wind.
This influence is noticeable since, at a moderate speed of 75 km/h (Fig. 6), A 30% forward wind compared to a similar no-wind case is therefore raising the fuel consumption of 18% (1.3 l) and a rearward wind of 30% is lowering the fuel consumption of 12% (0.9 l).

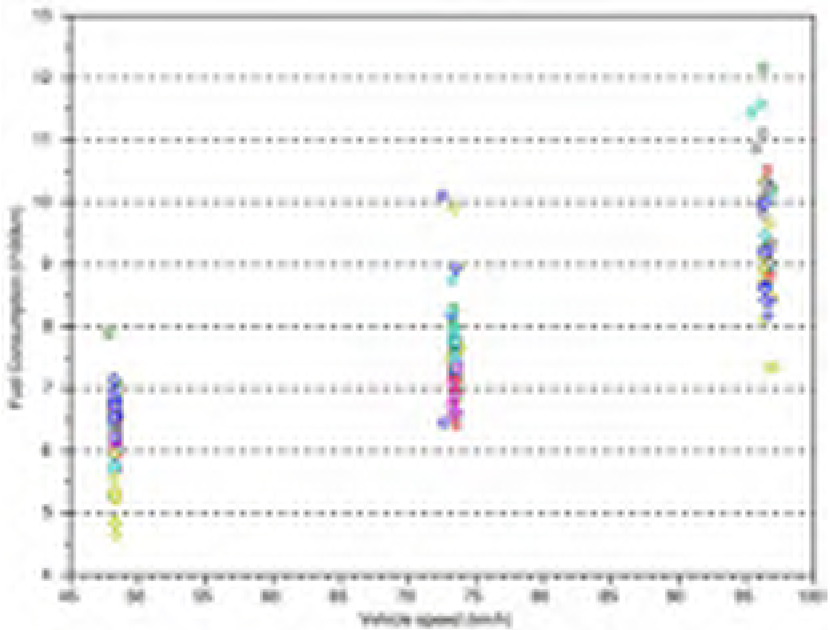

Figure 4 Ranges of fuel consumptions for three intended vehicle speeds of 50, 75 and 100 km/h (4th gear ratio)

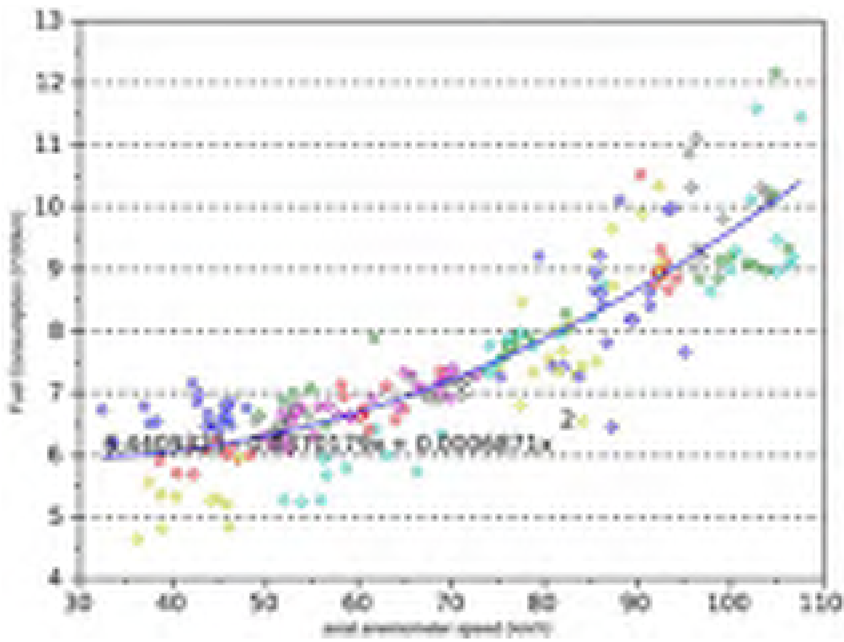

Figure 5 Fuel consumptions variations for the whole anemometer speed range (planned vehicle speeds of 50, 75 and 100 km/h ; 4th gear ratio)

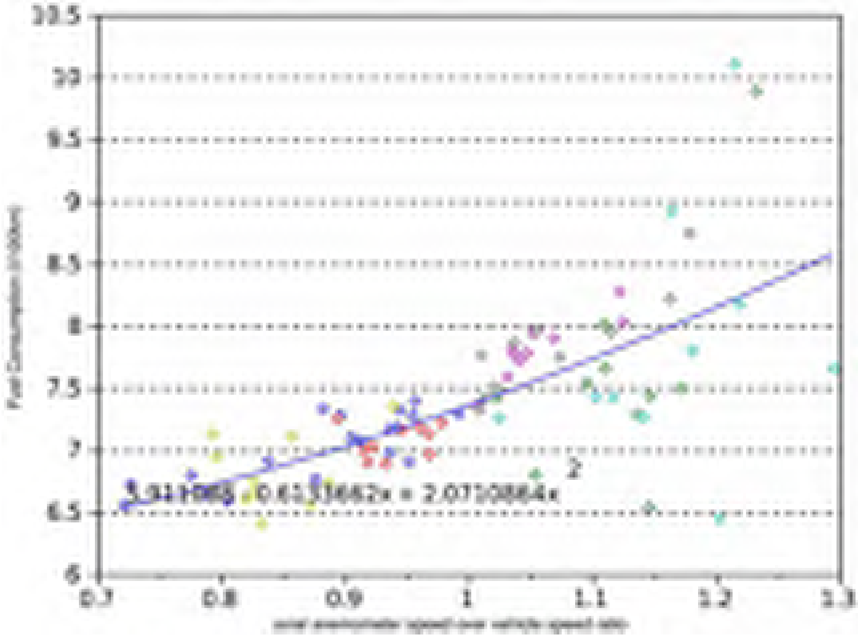

Figure 6 Influence of airflow speed over vehicle speed ratio (A/V ratio) on fuel consumptions (vehicle speed of 75 km/h ; 4^{th} gear ratio)

4 Conclusion

This work has been achieved in the general context of energy assessment of transportation. If considering the peak-oil hypothesis, energy for transportations could become less accessible and more expensive.

Models are used to assess the energy use phase of rail or road transportations, and, in this study, the aim is to evaluate if surrounding wind fields have to be taken into account in these models, in addition of own vehicle speeds.

In a first part, wind influence on trains consumptions have been evaluated. Numerical determination of aerodynamic coefficients for several wind and topological situations have been associated to atmospheric characteristics determined over a high-speed line. Simulation results show that, despite unfavorable conditions, with a train traveling at high speed in a weak wind field, wind influence can yet reach 5% of the aerodynamic forces power, compared to the without wind case. Moreover, for slightly more favorable conditions, a moderate train speed (about 45 m/s) and a slightly stronger wind speed (about 5 m/s) are leading to a computed wind influence of about 30% on the aerodynamic forces power, still while comparing to the without wind case.

In a second part, road experiments have been conducted with a medium-size passenger car equipped with an air flow meter and an oil flow meter, in addition of dynamical measurements as vehicle speed. Similarly to the railway study, wind influence is noticeable since, at a moderate speed of 75 km/h, a 30% forward wind compared to a similar no-wind case is therefore raising the fuel consumption of 18% and a rearward wind of 30% is lowering the fuel consumption of 12%. The non-symmetric aspect of these results can be imputing to the transversal wind field component, which should be investigated in a next step.

In conclusion, this work, based on full-scale experimental data and numerical simulations, points out that wind influence on total aerodynamic power is noticeable, both for railways or road transportation systems. Magnitude order of these influences is given, in the aim to justify the necessity to take into account wind fields for modeling use phase energy of these transportation systems.

References

[1] Esposito, T., Mauro, R., Russo, F., Dell'Acqua, G.: Speed prediction models for sustainable road safety management, Proc. Soc. and Behavioral Sc. 20, pp. 568–576, 2011.

[2] Tingvall, C., Stigson, H., Eriksson, L., Johansson, R., Krafft, M., Lie, A.: The properties of Safety Performance Indicators in target setting, projections and safety design of the road transport system, Accident Analysis and Prevention 42, pp. 372–376, 2010.

[3] Hubbert, M.K.: Nuclear energy and the fossil fuels, Presented at the spring meeting of the American petroleum institute, San Antonio, Texas, 1956.

[4] Gallagher, B.: Peak oil analyzed with a logistic function and idealized Hubbert curve, Energy Policy, 39, pp. 790–802, 2011.

[5] IEA (International Energy Agency): World Energy Outlook 2010, http://www.iea.org/weo/2010.asp, 2010.

[6] Ou, X., Zhang, X., Chang, S.: Scenario analysis on alternative fuel/vehicle for China's future road transport: Life-cycle energy demand and GHG emissions. Energy Policy, 38, pp. 3943 – 3956, 2010.

[7] Coiret, A., Orfila, O., Kane, M.: Concept of controllability: From pavement skid resistance to safety diagnostics on roads, RGRA 891, pp. 36-40, 2011.

[8] Orfila, O., Coiret, A., Do, M.T., Mammar S.: Modeling of dynamic vehicle–road interactions for safety-related road evaluation, Accident An. and Pr. 42, 1736–1743, 2010.

[9] Coiret, A., Vandanjon, P.-O., Bosquet, R., Jullien, A.: Energy consumption induced by operation phase of railways and road infrastructures, 2nd International Conference on Road and Rail Infrastructure, 7–9 May 2012, Dubrovnik, Croatia, pp. 693-699.

[10] Vandanjon, P.O., Coiret, A., Muresan-Paslaru, B., Fargier, A., Bosquet, R., Dauvergne, M., Jullien, A., François, D. & Labarthe., F: Practical guidelines for life cycle assessment applied to railways project, in: Proc. International Symposium on Life Cycle Assessment and Construction. Nantes, France, jul.2012.

[11] Khaled, M., El Hage, H., Harambat, F., Peerhossaini, H.: Some innovative concepts for car drag reduction: a parametric analysis of aerodynamic forces on a simplified body, J. Wind Eng. Ind. Aerodyn. 107-108, pp. 36-47, 2012.

[12] Tomasini, G., Cheli, F.: Admittance function to evaluate aerodynamic loads on vehicles: Experimental data and numerical model, J. of Fluids and Structures 38, pp. 92 -106, 2013.

[13] Seity, Y., Brousseau, P., Malardel, S., Hello, G., Bénard, P., Bouttier, F., Lac, C., Masson, V.: The AROME-France convective scale operational model, Mon. Wea. Rev., 139, pp. 976-991, 2011.

[14] Raghunathan, R.S., Kim, H.-D., Setoguchi, T.: Aerodynamics of high-speed railways train, Progress in Aerospace Sciences, 38, pp. 469 – 514, 2002.

IMPACT OF NEW BUILT ROUNDABOUTS ON ENVIRONMENTAL IN CITY OF VINKOVCI

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Abstract

By development of road traffic and all increasing demand for transport, new requirements to meet that same transport demands emerge. Many roads and intersections fail to meet the increasing demands of traffic and thus contribute to an unstable and inefficient traffic flow, increased number of traffic accidents, environmental pollution and the resulting adverse impact on the economy. Among many solutions, roundabouts have proved themselves in many cases to be one of the most successful. In this study the research was conducted on newly built roundabouts that replaced traffic light signalized and unsignalized intersections at two locations in the town of Vinkovci. Collected traffic and project data were analyzed and processed with the help of aaSIDRA analytic deterministic model in order to obtain comparison results for fuel consumption, waiting time and harmful exhaust emissions of CO_2, NOx, CO and HC for the past and present state of the intersection. After comparing the results, it was concluded that the new roundabouts have proved themselves as the best solution for reducing adverse effects of traffic and, as such, represent the future of further construction.

Keywords: roundabouts, simulation, waiting time, fuel consumption, emissions of harmful gases, CO_2*, NOx, CO, HC*

1 Introduction

The traffic is highly valued and necessary part of modern society, but its rapid expansion has been identified as a major cause of unwanted side effects. Traffic congestion makes cities less pleasant place to live in and reduces transport efficiency, thus increasing travel time, fuel consumption and driver stress. Organizing traffic represents an environmental, economic and political problem. It is important to establish an economical and efficient transport network while causing as little negative consequences as possible, particularly the negative influence of traffic on man and the environment. However, due to vehicle exhaust gases the traffic heavily influences the environment, thus representing one of the tasks that traffic engineers and technologists need to solve. Exhaust emissions from motor vehicles contain a wide range of pollutants, of which the most important are carbon monoxide (CO), nitrogen oxides (NOx), carbon dioxide (CO_2), sulfur oxides (SOx), hydrocarbons (CH) or volatile organic compounds (VOC) and particulate matter (PM) which, through their chemical composition, significantly affect air quality and human health. The amount of emissions of harmful exhaust gases vary depending on the total frequency of traffic, type of intersection control, behavior of participants and the characteristics of the vehicle .
The road intersections are places of deceleration and unnecessary traffic flow stops, resulting in an increased fuel consumption, lost time and creation of a certain amount of harmful exha-

ust gases. The roundabouts have appeared as a positive solution to reducing the negative traffic effects. When compared to other classical intersections, the intersections with roundabouts enable a steady stream of vehicles moving through without unnecessary stopping, unlike other types of leveled intersections where the flow of vehicles must stop and wait to be allowed free entry into the intersection. This problem mainly occurs at intersections containing traffic lights with an enforced traffic flow stops and changes in the signal plan. To check the effects of roundabouts on the environment in this study, we have analyzed the two newly constructed roundabouts that have replaced both the signalized and unsignalized intersection in the town of Vinkovci. For a comparison of the present and past state of the intersection we have used all of the important efficiency indicators such as fuel consumption, waiting-time and harmful exhaust emissions of CO_2, NOx, CO and HC, obtained by using aaSIDRA analytic deterministic model based on collected traffic and project data on the observed intersections. The research results obtained will serve as an answer to the question of which intersection type has more favorable environmental impact and represents the optimal solution for the future planning and organization of traffic.

2 An overview of previous researches

Previous studies directly related to the topic of the impact of an existing roundabouts on the environment and the economy are based on mutual comparison of leveled intersections via environmental (fuel consumption, harmful exhaust gases emissions) and traffic (waiting time, the number of unnecessary stops) criteria.

In a study conducted by the author Mustafa et.al (1993) it was concluded that there exists a direct relationship between the quantity of exhaust gases and the frequency of traffic at intersections regardless of the type of signaling installed. Conducted simulations have showed that both signalized and unsignalized intersections produce up to 50 % higher amounts of toxic emissions than the ones with roundabouts [1]. Várhelyi (2002) has examined the effects of small roundabouts that have replaced signalized and unsignalized intersections, on harmful exhaust emissions, fuel consumption, vehicle waiting time and the number of unnecessary vehicle stops. Roundabouts that have replaced signalized intersections have effectuated results in reducing vehicle waiting time for 11 seconds per vehicle as well as in the number of unnecessary stops, which declined from 63 % to 26 % of the total number of vehicles at the intersection. Emission of carbon monoxide (CO) was reduced by 29 %, nitrogen oxide (NOx) decreased by 21 % and fuel consumption per car has been reduced by 28 %. Roundabouts that have replaced unsignalized intersections haven't effectuated results with respect to reducing the quantity of harmful exhaust gases. Emission of carbon monoxide (CO) increased by 6 %, nitrogen oxide (NOx) increased by 4 %, and fuel consumption per car increased by 3 % [2]. Retting et.al. (2002) has conducted a survey in which he concluded that roundabouts that have replaced three unsignalized intersections have reduced waiting times by 13-23%, and the number of unnecessary stops by 14-37% [3]. On three unsignalized intersections remodeled into roundabout ones, Mandavilli et.al (2003) has proven a reduction in the harmful exhaust gases emission of carbon monoxide (CO) of up to 32 %, nitrogen oxide (NOx) of up to 34 %, carbon dioxide (CO2) of up to 37% and hydrocarbons (CH) of up to 42 % [4]. Sides et.al (2005) has proved that six classic types of intersections (three signalized and three unsignalized) refurbished into roundabouts, on an annual basis enables fuel savings of 67,142 liters and reductions of harmful exhaust emissions of CO_2, NOx, CO and HC by 179,440 kg [5]. The research conducted according to Bergh (2005) estimated that savings in waiting time of 62-72 % would be achieved should five out of ten signalized intersections get reconstructed in the roundabout ones, which represents the approximate equivalent of 325 000 hours of waiting time in reduced delays at the annual level [6].

3 Collecting, processing and display of traffic data

For a comparison of fuel consumption, waiting time and harmful exhaust emissions of CO_2, NOx, CO and HC for the past and present states of the selected intersections, we chose two intersections with roundabouts that have replaced one signalized and one unsignalized intersection in the town of Vinkovci. The intersection with roundabout which has replaced the signalized intersection (V2) is located on the corner of Kneza Mislava street – Lapovačka street – Ivana Gorana Kovačića street and Ljudevita Gaja street, while the unsignalized intersection (V1) which was replaced by the intersection with roundabout is located on the corner of Hrvoja Vukčića Hrvatinića street and Hansa-Dietricha Genschera street. Figure 1. shows the present and past state of the intersection at the observed locations.

Past state at the V_1 location

Past state at the V_2 location

Present situation at the V_1 location

Present state at the V_2 location

Figure 1 Display of the present and past state at the intersection on V_1 and V_2 locations

To collect traffic data via video recording on mentioned V1 and V2 roundabouts sites, we used sports video camera Prestigio RoadRunner 700x for its technical characteristics and simplicity (compact size, wide angle coverage, autonomy of use, high resolution of full HD recording and easy video processing on the PC). Video recording of traffic at roundabouts was carried out during the working day on December 22nd 2013, over a four hour peak period (6:00 AM to 8:00 AM and 3:00 PM to 5:00 PM). Videos were reviewed manually and traffic data was collected on the type of vehicles (flow structure) and the number of vehicles flowing through the roundabout and approach (due to low frequency, the pedestrian traffic was not analyzed). The collected traffic data from morning and afternoon peak loads are reduced to one peak hour. At the V1 location, the peak hour for the morning period from 6:00 AM to 8:00 AM is located in the time interval between 6:30 AM and 7:30 AM, and on the V2 location in the time interval between 7:00 AM and 8:00 AM. The peak hour for the afternoon period from 3:00 PM to 5:00 PM is located in the time interval between 4:00 PM and 5:00 PM for the V1 location, and for the V2 location in the time interval between 3:00 PM and 4:00 PM. Table 2 shows the peak hour for the morning and afternoon period at the V1 location and Table 3 shows the peak hour for the morning and afternoon period at the V2 location. The collected traffic data are implemented in the aaSidra analytic deterministic model for simulating the present situation, and the same traffic data were used to simulate the past state at the intersections.

Table 1 The peak hour for morning and afternoon period at V_1 location

LOCATION V1	APPROACH 1					APPROACH 2					APPROACH 3					APPROACH 4					
	1.2	1.3	1.4	1.1	Σ	2.3	2.4	2.1	2.2	Σ	3.4	3.1	3.2	3.3	Σ	4.1	4.2	4.3	4.4	Σ	Σ
6:30-6:45	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6:45-7:00	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7:00-7:15	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7:15-7:30	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
ukupno	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
phf	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
LV	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
heavy v.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
motorcycle	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
bus	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
bicycles	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
LOCATION V1	APPROACH 1					APPROACH 2					APPROACH 3					APPROACH 4					
	1.2	1.3	1.4	1.1	Σ	2.3	2.4	2.1	2.2	Σ	3.4	3.1	3.2	3.3	Σ	4.1	4.2	4.3	4.4	Σ	Σ
16:00-16:15	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
16:15-16:30	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
16:30-16:45	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
16:45-17:00	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
ukupno	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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phf	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
LV	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
heavy v.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
motorcycle	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
bus	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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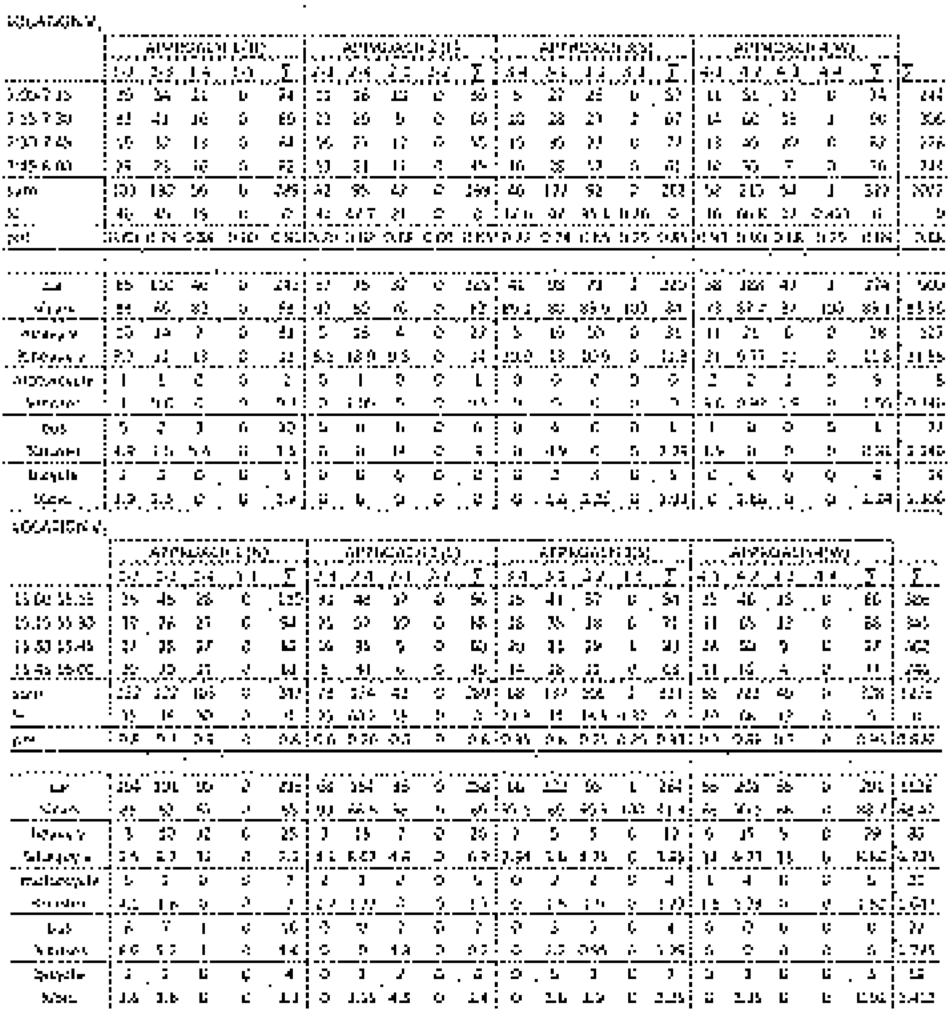
bicycle	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table 2 The peak hour for morning and afternoon period at V_2 location



From the design documentation for the observed intersections, provided to us by the Town of Vinkovci, we've gathered the project sizes of: outer radius, that is the intersection's inner radius, the width of the circular roadway, the width of the entrance and exit parts of approach , the width of the approach lane, the width of the isles or dividers in approach , the entrance angle , the entrance and exit's round off, the length of the extension, sharpness of the extension, lateral circular inclination of the track and drainage, which were implemented in the aaSIDRA analytic deterministic model.

4 aaSIDRA analytic deterministic model

In making of this study we've used aaSIDRA analytic deterministic model (signalized and unsignalized Intersection Design and Research Aid) developed by Akcelik and Associates for the design and evaluation of various forms of at-grade intersections (circular, signalized and unsignalized) [7].
On basis of certain input data, the program calculates output values and, among other things, calculates the level of service at the intersection, waiting time, fuel consumption, environmental impact, and ultimately the very viability of the intersection. Depending on the structural characteristics of the intersection it is possible to switch between the types of intersections depending on user requirements, and therefore it is possible to analyze signalized, unsignalized, circular, four-spoke and T-shaped intersections. It is as well possible to set the main ways and alternate the signaling at the intersection, and thus analyze a particular option of an intersection arrangement.
For the purposes of this study we used the input data (project- design elements intersections and traffic data) in aaSIDRA program through which we've reached the output data (waiting time, harmful exhaust emissions, fuel consumption). To estimate fuel consumption and harmful exhaust emissions, aaSIDRA program uses four basic sequences of driving through an intersection, acceleration, deceleration, idling and driving (see Figure 2) for each driving lane, while taking into account the light and heavy vehicles used to model the consumption and emissions of harmful exhaust gases.

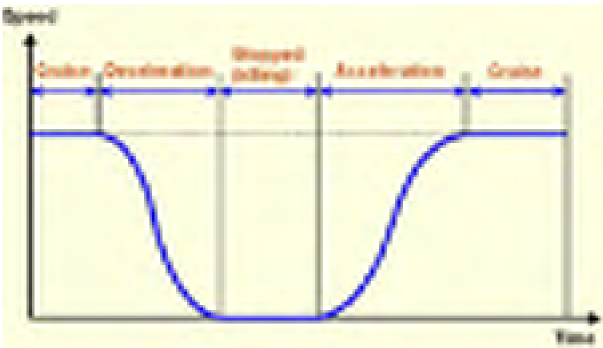


Figure 2 Four basic driving sequences

5 Output data for the present and past state of the intersection

aaSIDRA program output data for the quantities of harmful exhaust emissions of CO_2, NOx, CO and HC, the waiting time and fuel consumption for the entire circular and unsignalized intersection for morning and afternoon peak hour on the V1 location is shown in Table 3. Table 4. shows all output data for the entire circular and signalized intersection at the V2 location for morning and afternoon peak hour.
From Table 3. it can be concluded that there are several significant changes in relation to the comparison of the past and present state of the intersection. Quantities of carbon monoxide (CO) have been reduced by 6% to 10%, nitrogen oxide (NOx) by 6.7%, while the amount of hydrocarbons (HC) increased by 1.5% to 3.3%, and the waiting time for 17% to 33%. According to Table 4., it can be concluded the roundabout that has replaced the signalized intersection, has in all output parameters significantly reduced the waiting time at the intersection, as well as the amount of harmful exhaust emissions and unnecessary fuel consumption.

Table 3 Display of output dana for morning and afternoon peak hour on the V_1 location

Location V1 morning results 6:30 AM – 7:30 AM				
Pollutant	Roundabout	Unsignalized intersection	Difference	Percentage (%)
Carbon dioxide (CO2) (kg/h)	212.7	214.7	-2 (kg/h)	-0.93
Carbon monoxide (CO) (kg/h)	12.3	13.2	-0.9 (kg/h)	6.82
Hydrocarbons (HC) (kg/h)	0.273	0.269	0.004 (kg/h)	1.49
Nitrogen oxide (NOX) (kg/h)	0.471	0.505	-0.034 (kg/h)	-6.73
Fuel consumption (l/h)	84.8	85.4	-0.6 (l/h)	-0.70
Waiting time (s)	9.9	8.4	1.5 (s)	17.86
Location V1 afternoon results 3:00 PM – 4:00 PM				
Pollutant	Roundabout	Unsignalized intersection	Difference	Percentage (%)
Carbon dioxide (CO2) (kg/h)	180.2	175	5.2 (kg/h)	2.97
Carbon monoxide (CO) (kg/h)	8.72	9.66	-0.94 (kg/h)	-9.73
Hydrocarbons (HC) (kg/h)	0.248	0.24	0.01 (kg/h)	3.33
Nitrogen oxide (NOX) (kg/h)	0.367	0.367	0.00 (kg/h)	0.00
Fuel consumption (l/h)	71.8	69.8	2 (l/h)	2.87
Waiting time (s)	8.8	6.6	2.2 (s)	33.33

Table 4 Display of output dana for morning and afternoon peak hour on the V_2 location

Location V1 morning results 6:30 AM – 7:30 AM				
Pollutant	Roundabout	Unsignalized intersection	Difference	Percentage (%)
Carbon dioxide (CO2) (kg/h)	252.8	264.6	-11.8 (kg/h)	-4.46
Carbon monoxide (CO) (kg/h)	16.64	18.55	-1.91 (kg/h)	-10.30
Hydrocarbons (HC) (kg/h)	0.32	0.347	-0.027 (kg/h)	-7.78
Nitrogen oxide (NOX) (kg/h)	0.615	0.666	-0.051 (kg/h)	-7.66
Fuel consumption (l/h)	100.5	105.2	-4.7 (l/h)	-4.95
Waiting time (s)	12.2	14.6	-2.4 (s)	-16.44
Location V2 afternoon results 3:00 PM – 4:00 PM				
Pollutant	Roundabout	Unsignalized intersection	Difference	Percentage (%)
Carbon dioxide (CO2) (kg/h)	260	267.5	-7.5 (kg/h)	-2.80
Carbon monoxide (CO) (kg/h)	16.12	17.04	-0.92 (kg/h)	-5.40
Hydrocarbons (HC) (kg/h)	0.369	0.389	-0.02 (kg/h)	-5.22
Nitrogen oxide (NOX) (kg/h)	0.585	0.605	-0.02 (kg/h)	-3.31
Fuel consumption (l/h)	103.6	106.6	-3 (l/h)	-2.81
Waiting time (s)	14.1	17.9	-3.8 (s)	-21.23

6 Conclusion

According to the stated in this study, it can be concluded that the newly built circular intersection replacing the unsignalized one has partly contributed to the reduction of exhaust emissions, fuel consumption and waiting time. Circular intersection that replaced the signalized intersection has significantly contributed to the reduction of exhaust emissions, fuel consumption and waiting time. It is therefore necessary to undertake additional research related to the reconstruction of the existing unsignalized intersections into signalized or circular ones, that is it is necessary to develop a scientific methodology for justification of reconstructing the existing unsignalized and signalized intersections into circular intersections.

References

[1] Mustafa, S., Mohammed, A., Vougias, S.: Analysis of Pollutant emissions and Concentrations at Urban Intersections, Institute Of Transportation Engineers, Compendium Of Technical Papers, 1993.

[2] Varhelyi, A.: The effects of small roundabouts on emissions and fuel consumption: acase study, Elsevier Science Ltd, Sweden, 2002.

[3] Retting, R. A., Luttrell, G., Russell, E. R.: Public opinion and traffic flow impacts of newly installed modern roundabouts in the United States. ITE Journal, 2002, vol. 72, no. 9, pp. 30 – 32.

[4] Mandavilli, S., Russell, E. R., Rys, M.: Impact of Modern Roundabouts on Vehicular Emissions. Kansas State University, USA, 2003.

[5] Sides, K., Seals, J., Wallwork, M.: Conversion of Three Signalized Intersections and Three Stop-controlled Intersections to Modern Roundabouts on Cleveland Street in Clearwater, Florida. Impact Study. TRB-National Roundabout Conference, USA, 22-25, May, 2005.

[6] Bergh, C., Retting, R. A., Meyers, E. J.: Continued reliance on traffic signals: the cost of missed opportunities to improve traffic flow and safety at urban intersections. Arlington, VA: Insurance Institute for Highway Safety., USA, 2005.

[7] aaSIDRA User Guide. AKCELIK and ASSOCIATES. Pty Ltd, Melbourne, Australia. (2011).

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ISSUES RELATED TO THE IMPACT OF NOISE AT AT-GRADE INTERSECTIONS

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Abstract

The article addresses the issues of determining the impact of noise at at-grade intersections. While noise levels on open road sections may be determined relatively unambiguously, at intersections the noise pollution levels are relatively difficult to identify. It is, particularly, in urban areas where this becomes an essential problem that significantly affects the impact of designed transport solutions on their surroundings. At the same time, it is obvious that this type of noise pollution in the road network is one of the highest ever because of the vehicle movement characteristics. Therefore, a realistic evaluation of intersections is impossible without the identification of complex noise pollution and its distribution in the intersection area. This was done using the AIMSUN micro-simulation software environment. The generated result reflects both the traffic and layout characteristics of individual intersections. Traffic noise pollution levels were determined through the use of dot array.
Noise pollution is identified primarily with regard to the impact of its adverse effects on human health. This evaluation takes into account mainly the emergence or deterioration of some diseases. (also so-called lost years of life, the costs of hospital stays, etc.). The evaluation method of these effects is based on standards also used in EU countries; the methodology is also applied in the HDM-4 system. Thus, it allows determining a specific impact of individual noise levels on adjacent areas and the population. External costs of noise pollution may also be quantified. The paper gives a detailed account of the methodology for the determination of average noise pollution, its external costs and the dependence of these parameters on the shape of an intersection and its entry traffic volumes. It represents one of the basic criteria used for a complex assessment of intersections.

Keywords: noise emissions; intersection; HDM-4; traffic micro-simulation; external costs

1 Introduction

The impact of traffic noise is becoming one of the important aspects considered in the assessment and optimization of proposed traffic solutions, particularly in urban areas. The existing methodologies for detecting traffic noise are primarily designed to determine the noise level on open road sections. A problem, however, arises in urban areas where the main limiting factor of the road network are at-grade intersections where the possibilities of current methodologies are strictly limited. While the noise generated by car traffic may be identified without significant problems for existing design solutions, for newly built intersections or reconstructions of existing intersections this problem becomes far more complicated. The methodology described in the article uses an alternative procedure to identify the impact of traffic noise generated at at-grade intersections. However, the methodology is primarily designed to serve for comparative assessments of alternative designs of an intersection, not for a predictive identification of actual noise emissions. It is based on existing procedures exploiting the potential of the Aimsun micro simulation software from which the input data for the assessment of noise emissions are generated. The resulting data of traffic noise levels may then be used as part of a complex assessment of alternative designs of an at-grade intersection.

2 Traffic noise and its impacts

2.1 Basic parameters of noise emissions

The existing methodologies for the identification of noise effects on the human body use the values of L_{eq} (equivalent continuous sound pressure level). This is a fictitious level of steady noise pressure which has the same effects on the human organism during a reference time as variable noise. The total acoustic energy of variable noise for a reference time must be equal to steady noise with a constant level L_{eq}. Therefore, not a simple average, but mean energy applies in this case. The A-weighted filter, which represents the inverse sensitivity curve of the human ear, is used to measure noise from road traffic. This filter is built-in in measuring instruments, and the values corrected by this filter get the subscript "A". The value obtained is then called L_{Aeq} (equivalent continuous A-weighted sound pressure level).
Noise does not propagate linearly in space. The logarithmic scale is commonly used for the description of noise propagation. The effect of the noise source intensity and distance from the source is shown in Fig. 1.
Noise impacts are naturally regulated by the legislation. The legislation currently in force (Czech Republic) defines public health noise limits in protected outdoor spaces of structures. The $L_{Aeq,T}$ for noise from road traffic is detected for the whole day ($L_{Aeq,16}$) and night time ($L_{Aeq,8}$). Public health exposure limits are set as the arithmetic sum of the equivalent sound pressure level $L_{Aeq,T}$ = 50 dB and a correction considering the type of protected space and the day time or night-time. Table 1 presents public health exposure limits for the protected outdoor space of other structures in the day time, depending on the type of road.

Table 1 Public health exposure limits according to the type of road (CR)

Road Type	Public health exposure limit [dB]	
	Day (6-22 h)	**Night (22-6 h)**
Main roads	60	50
Minor roads	60	50
Other types of roads	55	45
Old noise values	70	60

For the usage in our methodology the value L_{Aeq} is not ideal. The methodology should be able to determine the complex traffic noise impact, nevertheless value L_{Aeq} defines the value for peak volumes mainly. For our purpose value L_{dvn} will be more suitable, because it takes into account volume distribution during the day.

Figure 1 The effect of noise source intensity and distance from the source

2.2 Noise from car traffic

The basic sources of noise from car traffic are the following:
- noise of the engine (drive unit)
- noise arising during the contact of wheels with the pavement (rolling), or also rolling noise
- aerodynamic noise (bypassing)
- noise of transmission and distributor gear
- noise of the exhaust system
- noise of brakes
- noise of impacts and shocks of the load.

Engine noise is the primary source of noise prevailing particularly at lower vehicle speeds. For passenger cars, it dominates at speeds of up to 30 km/h, while for heavy vehicles at speeds of up to 50 km/h. “Total noise emissions from vehicles” are currently regulated by the EU limit of 74 dB for passenger cars and 80 dB for heavy vehicles.
Noise levels are further affected by other factors (i.e. traffic volumes, traffic flow composition, vehicle speed, pavement surface etc.) The resulting noise level at a given site is affected by numerous factors, some of which are under our control while others are steady and invariable.

3 Noise emission model in intersection areas

3.1 Usability of existing methods at intersections

It is obvious from the above facts (2.2) that the noise generated in the area of intersections is mainly generated by the vehicle drive unit due to the expected lower driving speed of vehicles. For this reason, while considering noise emissions we must also take into account other factors than just the vehicles driving speed. This may be mainly apparent at intersections with traffic lights where a traffic flow starts moving all at once in accordance with a signal plan. The noise generated by the engine during the start depends on the engine rpm, not on the driving speed. From this perspective, the application of existing static methodologies at intersections without their major modifications is practically useless. Another problem of intersections is the identification of the factor that can be measured, or at what point is can be measured. Complex assessment based on micro-simulations would have to take into account information on the movement of individual vehicles and, applying the principle of the detection of the sound pressure level generated by these vehicles, establish equivalent sound pressure levels.
The above described methodology would set extreme computational demands and would be unnecessary for our purposes. Therefore, our starting point is the current applicable met-

hodology for calculating noise levels from car traffic, which was calibrated and modified for our needs. In other words, proposed methodology combines current static calculations with technical information of vehicles used in evaluated micro-simulation model, i.e. on evaluated intersection. The calibration of the microscopic model itself was based on the output of a classical noise assessment of this model intersection performed in the HLUK+ specialized programme as well on the comparison of measured in-situ values with the values from existing micro-simulation model.

3.2 Design of a methodology of area detection of noise emissions

The designed methodology is based on the fact that the noise level detection at a specific point of intersections is not a relevant value indicating the overall area noise emissions generated by traffic in the intersection area. Thanks to the possibilities of the Aimsun micro simulation software, however, we are able to express the sound pressure value at each point in the vicinity of an intersection. With a view to noise characteristics, valid limit values are detected at a distance of 7.5 m from the axis of the adjacent lane. The values were detected up to the distance of the branches (100 m) from the intersection boundary (Fig. 2). The area delimited by these boundaries is relevant for the detection of average noise emissions generated by the intersection in the form of the $L_{dvn,int}$ value (equivalent continuous sound pressure level in the intersection area).
For the purposes of computations limitation, the delimited area was subdivided by a square grid with squares 2x2 m. While a 2 m difference would have a significant impact on the accuracy of the result in point detection of noise levels, interpolation methods may be used to identify the values with sufficient accuracy in area detection. The accurate $L_{dvm,i}$ value is detected at each intersection of the axes of the square grid to be used as an input value for the identification of the resulting value $L_{dvn,int}$. The distribution of individual $L_{dvn,i}$ values also defines the range of respective equivalent sound pressure levels. Due to the nature of noise, however, it is also necessary to take into account the negative impact of individual noise levels on the surroundings, namely on the health of the population influenced with traffic noise.

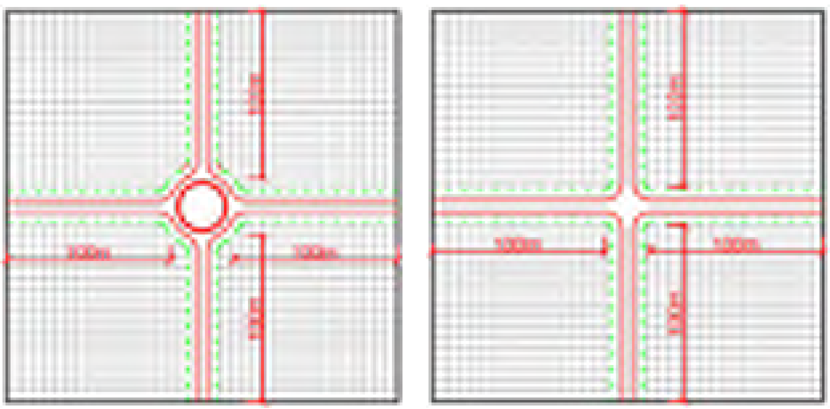


Figure 2 Area detection of sound pressure levels at a roundabout and a four-way intersection

The existing HDM-4 methodology, which determines the prices of external costs of noise exposure, was used to identify these impacts (Table 2). The essential figure in the context of our assessment is not the real quoted price, but the ratio between individual noise levels.
The $L_{Aeq,int}$ value is calculated from the following formula (1):

$$L_{dvn,int} = \frac{\sum_{i=1}^{n} L_{dvn,i} * A_i * C_i}{\sum_{i=1}^{n} A_i * C_i} \tag{1}$$

where:

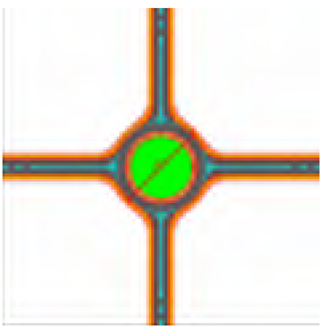
$L_{dvn,i}$ the L_{dvn} value at the i-th point of a square grid [dB];
A_i specific area corresponding to the $L_{dvn,i}$ value [2×2 m = 4 m^2];
C_i price of external costs of noise exposure [€].

Table 2 External costs of noise exposure (2010 – per person and year)

L_{dvn} [dB]	Cost [€]	L_{dvn} [dB]	Cost [€]	L_{dvn} [dB]	Cost [€]
53	30.6	62	126.2	71	277.3
54	42.1	63	135.8	72	294.6
55	51.6	64	147.3	73	313.7
56	63.1	65	156.9	74	330.9
57	72.7	66	168.3	75	348.1
58	84.1	67	177.9	76	382.5
59	93.7	68	187.4	77	399.7
60	105.2	69	198.9	78	417.0
61	114.7	70	208.5	79	436.1

3.3 Methodology application to a model example

The selected model example was a single-lane roundabout with four perpendicular branches formed by a two-way two-lane road. The assessment involved overall traffic volumes carried by the intersection ranging between 600 – 2800 veh/h, with a symmetrical distribution of traffic volumes into individual branches (i.e. in the ratio 1:1:1:1), intersection movements in the ratio 1:2:1 (left/through/right), the percentage of heavy vehicles being 4% in all branches. The terrain is considered as non-reflective, the longitudinal slope being 0%. The assessed intersection and individual traffic volumes are displayed in Fig. 4.



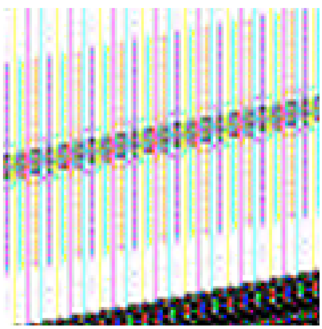


Figure 3 Assessed intersection and distribution of approach traffic volumes

The resulting values for individual approach traffic volumes are presented in the text and graphic format in Figure 5. An assessment of a four-leg four-way intersection with traffic lights was performed for comparison.

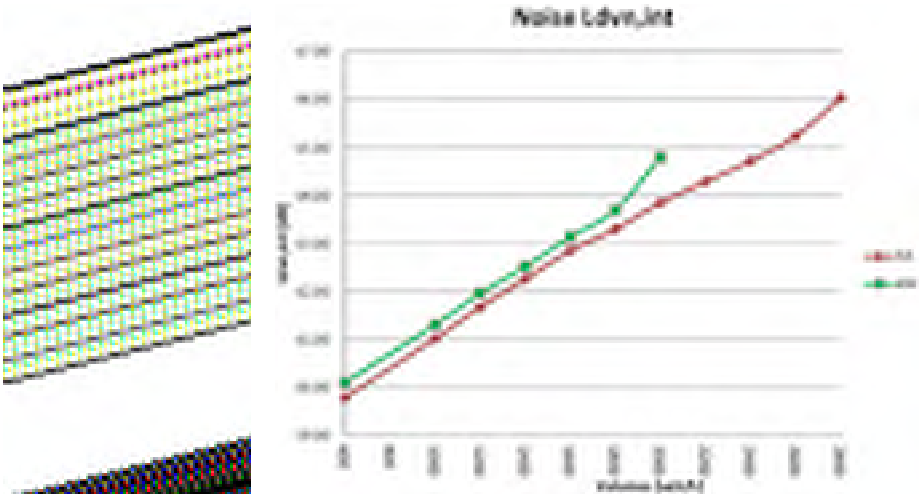


Figure 4 Assessed intersection and distribution of approach traffic volumes

Figure 5 also clearly shows the rapid growth in noise emissions after the intersection capacity was exceeded (for RA over 2600 veh/h, for 4W over 1800 veh/h). The assessment of the values once the intersection capacity was exceeded is pointless as computational methods operate with a free traffic flow and, therefore, congestion generated noise is subject to a considerable error. The difference in the distribution of individual noise levels is evident from Figure 6 showing a noise map for a model intersection at the overall traffic volumes carried of 1000 veh/ h (left) and 2400 veh/h (right).

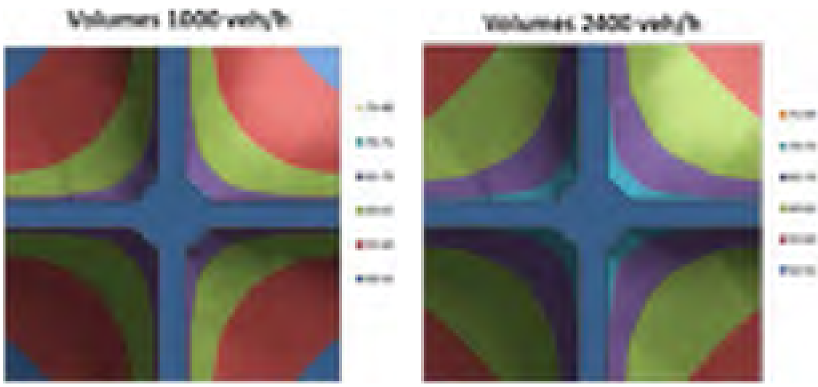


Figure 5 RA noise maps for 1000 veh/h and 2400 veh/h

4 Future application potential

The above procedure provides sufficiently valid data for the assessment of alternative design solutions of intersections under identical conditions. The application options are offered primarily to designers, traffic engineers and municipalities. No matter what the resulting data are used for: as input data for multi-criteria analysis or for a separate assessment in terms of noise emissions, we may say that the final evaluation will be valid and will identify the differences between individual design solutions considered. This methodology may appear more problematic in the determination of the actual impact of traffic on the surroundings. Here, a relatively large number of factors that can significantly affect the result enter the assessment (such as terrain reflectance, type and configuration of surrounding developments, density of population, etc.). Despite the fact that the calibration procedures verifying model results provided results within acceptable tolerance limits (ca up to 0.6 dB), sufficient accuracy of the model cannot be unambiguously confirmed. On the other hand, it must be added that this calculation model is sufficiently accurate and more suitable for the assessment of the impacts of car traffic in the area of intersections than the design methodology without any corrections. An even more accurate model results would be achieved only in the case of its interconnection with traffic engineering micro- and nano-simulation software where the movement of individual vehicles would have to be continually tracked and their impact on noise emissions defined in surrounding area.

References

[1] ČNI, ČSN 73 6102 – Design of road intersections, ČNI, Praha, ČR, 2001.

[2] Charles University – Center for environmental issues, Methodology of traffic noise evaluation, project MD ČR CG712-11-520, CDV, Prague, 2012

[3] Čihák, M.: Update of system for economical effectivity evaluation of road structures – HDM-4, Silniční obzor 2013, Vol. 77, pp. 120-124. ISSN 0322-7154, 2013.

[4] Jahandar, N., Hosseinpour, A., Sahraei, M.A.: Traffic Noise under Stop and Go Conditions in Intersection – A Case Study, World Academy of Science, Engineering and Technology vol. 62, pp. 465-468, ISSN 2010-376X, 2012.

[5] Decký, M.: Noise Pollution from Roundabout Traffic in the Outer Environment of Built-up Areas of Towns, Perners contacts, electronic paper of University of Pardubice, IV. Volume, ISSN 1801-674X, pp. 53-68, 2008.

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THE IMPACT OF INTERSECTION TYPE ON TRAFFIC NOISE LEVELS IN RESIDENTIAL AREAS

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Abstract

Today road traffic noise represents one of the most serious environmental problems in urban areas. It is well known that the specific deceleration and acceleration dynamics of traffic at road intersections can cause different noise levels than free-flow traffic on open road segments. Also, each intersection type has variously distributed sections with different traffic flow conditions: constant speed, stop and go, deceleration and acceleration. The aim of research described in this paper is to establish whether the design of road intersection has influence on traffic noise emissions and to establish which intersection type is most suitable for application in urban areas, where residential buildings are often placed directly next to the noise source – in this case, the intersection. Models used for noise modeling in this research consist of two identical intersecting roads connected as follows, by mini roundabout and by intersection with or without traffic lights. Traffic noise calculations were conducted by means of specialized noise prediction software LimA using modified static noise calculation method RLS 90. Accuracy of roundabout noise model described in this paper had been verified in previous studies by comparison of measured noise levels and calculated noise levels on a number of urban roundabouts.

Keywords: intersection type, road traffic noise levels, traffic flow, noise modeling

1 Introduction

Today, in time of rapid technological progress and urbanization, and due to everyday exposure of population to high noise levels, negative effects of noise from various sources are increasingly noticed. Road traffic noise comprises an appreciable segment of total share of disturbance caused by noise in residential areas and therefore engineers are constantly working on finding new, more efficient and also most cost-effective protection measures that would reduce its levels to those prescribed by regulations.
It is well known that the specific dynamics of traffic flow at road intersections can influence local noise emission. Considering the fact that each intersection type has variously distributed sections with different traffic flow conditions (constant speed, stop and go, deceleration and acceleration) and the fact that vehicles under stop-and-go conditions produce more noise compared to traffic at a constant speed [1], it can be stated with certainty that traffic noise levels depend on intersection geometry and applied traffic regulation.
A problem with increased noise levels in the vicinity of intersections especially occurs in urban areas where residential buildings are placed right next to the road. Minor city streets are usually intersected by regular crossings with or without traffic lights and, lately, by roundabouts. In this paper, traffic noise modeling of these three most common types of urban road intersections is described. The road traffic noise analysis was conducted by means of specialized noise prediction software LimA using static German RLS 90 model, modified for the use in local

traffic conditions. Each testing model contains segments with different traffic flow conditions that correspond to real driving pattern through particular intersection type. The main goal of this study is to present the influence of certain intersection type on traffic noise emissions.

2 Traffic noise in the vicinity of road intersections

Generally, traffic noise situation can be determined by: field measurements, calculations conducted by means of specialised noise prediction software (noise modeling) or by combination of both measurements and calculation. For the purpose of this research, noise modeling procedure was carried out – noise modeling is most suitable method for comparison of various noise model scenarios, which is in this case a comparison of noise emissions in the vicinity of different road intersection types.

2.1 Road intersection noise modeling

Temporal and spatial variations in vehicle speed have a substantial impact on traffic noise emissions [2]. Depending on the way noise prediction models account for traffic flow, dynamic effects are more or less accurately captured. In static noise models, roads are divided into sections where the traffic flow is considered smooth and homogeneous. Since this assumption does not hold in the vicinity of intersections, some models include a propagation correction term.
In this research noise calculations were conducted by means of specialized software LimA using modified static German RLS 90 model. RLS 90 model includes the influence of road intersections on traffic noise levels by introducing propagation correction term for intersections with traffic lights, for up to a distance of 100 m from two nearest intersecting lane axis [3]. To capture the difference between noise levels of various intersection types it was necessary to make certain method modifications. Accuracy of modified RLS 90 noise model described in this paper had been verified in previous studies by comparison of measured noise levels and calculated noise levels on a number of urban roundabouts in Zagreb [4].

2.2 Intersection noise models

In order to investigate the impact of intersection type on traffic noise levels, three different noise models were designed:
· four leg intersection without traffic lights (priority to the right),
· four leg intersection with traffic lights and
· four leg mini roundabout.

Approaching legs of all testing models consist of identical two-lane two-way urban roads connected at an angle of 90°. Priority to right model and model with traffic lights have the identical intersection geometry. Mini roundabout was designed with circulatory roadway radius which occupies the same area as the first two regular intersections. The source of the traffic noise was situated at 0,5 m above the road surface in each lane axis and it was presumed that input traffic load was divided equally into all driving directions. Due to the comparability of results another assumption was that vehicles in roundabout drive only right, straight and left, excluding U-turn. In modified RLS 90 model used in this research each lane axis is divided into segments with different traffic flow conditions (constant speed, stop and go, deceleration and acceleration) in accordance with real driving situation (Figure 1). Specific dynamics of traffic were defined by the average traffic speed of certain segment, and stopping and queuing by introducing traffic light position on each stopline and the entrance on the crossing area i.e. circulatory roadway. Identical input data on other noise emission parameters (vehicle speed, road surface type, longitudinal slope) and identical noise propagation parameters were defined for all testing models.

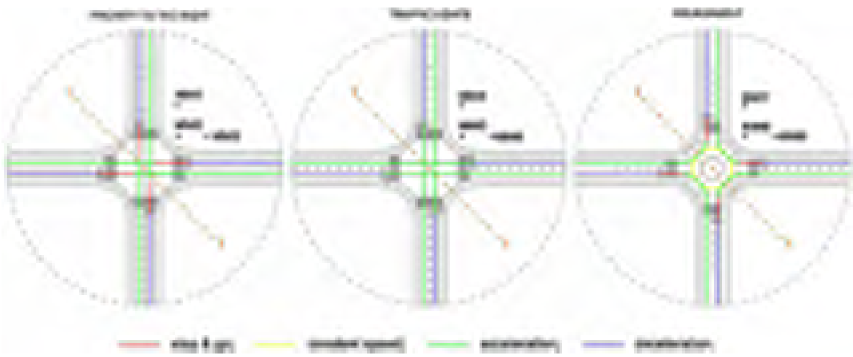


Figure 1 Intersection noise models

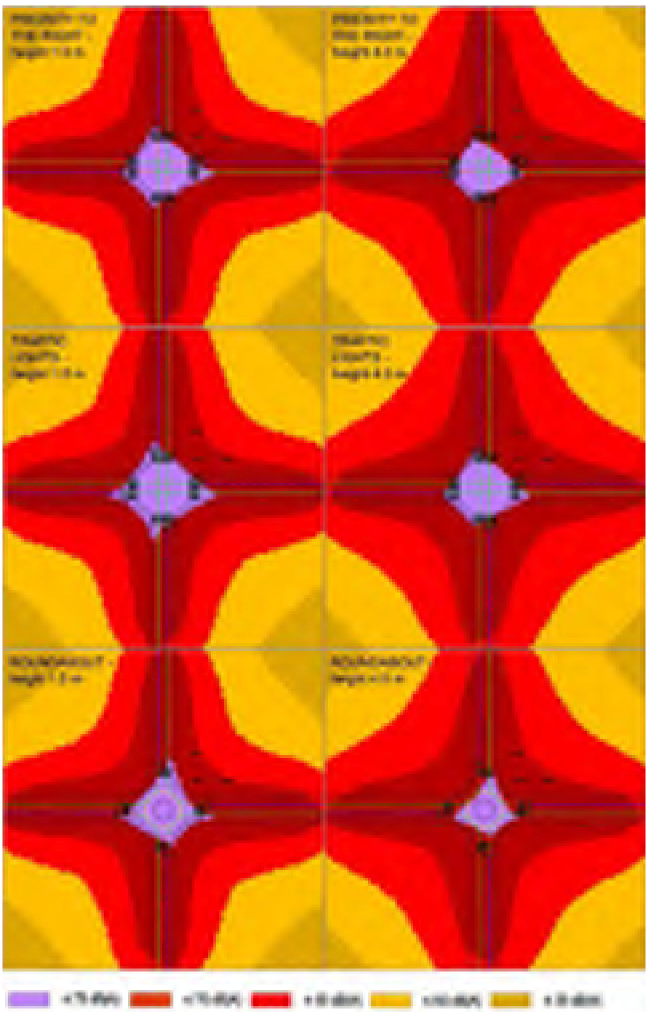


Figure 2 Predicted noise levels for period “day” for different intersection types

2.3 Noise prediction results

Traffic noise calculation procedure was carried out by means of specialized noise prediction software. All noise mapping simulation software packages have facilities to produce noise maps where receivers are arranged in a grid pattern following the terrain in a set height. In this research, noise prediction results were presented using noise maps (Figure 2) created at grid points defined by regulations [5] (grid 10x10 m at height of 1.5 and 4 m) and diagonal cross sections (Figure 3) showing predicted noise levels in intersection area to a height of 10 meters above driving surface.

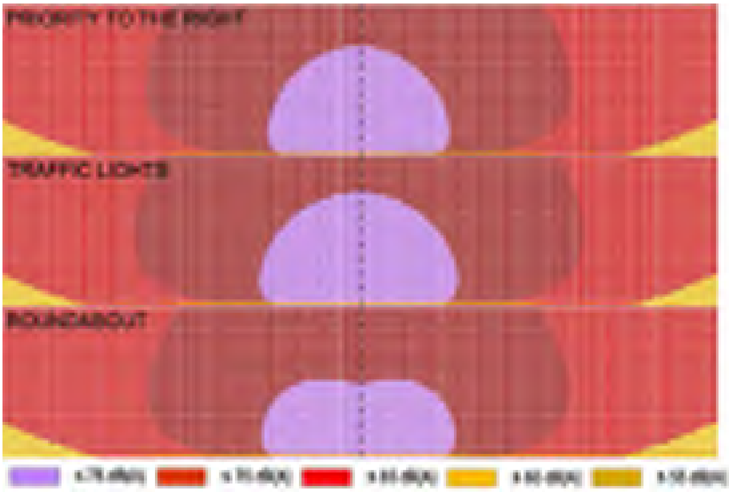


Figure 3 Predicted noise levels for period "day" at intersection diagonal cross-sections (1-1) in width of 50 meters

In Figure 4 comparison of resulting noise contours of all testing models is presented (noise results of all three intersections were folded through the centre of their axes).

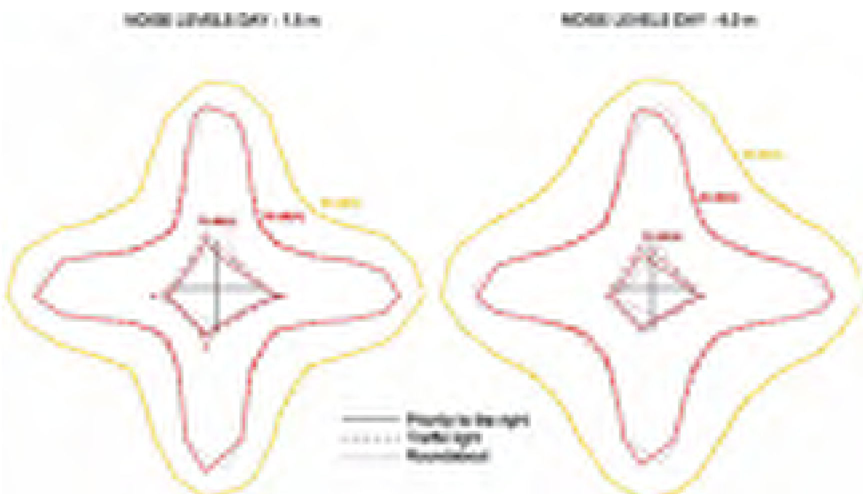
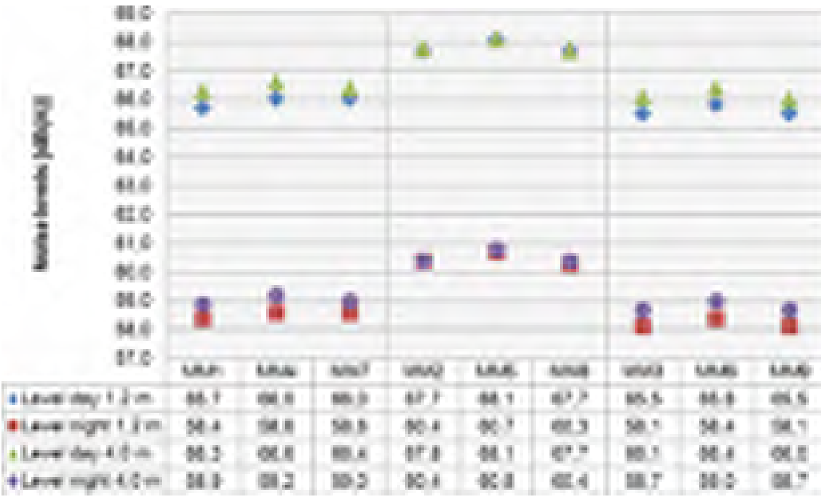


Figure 4 Comparison of noise prediction results

Beside the grid points, noise calculations were conducted in three free field receptors positioned at same locations for all testing models – at height of 1.2 m and 4.0 m, 7.5 meters from the axis of the lateral lane at entry and exit lanes and next to the intersection area. The results of these calculations are presented in Table 1.

Table 1 Traffic noise levels in free field receptors



2.4 Discussion

Noise prediction results showed that differences in predicted traffic noise levels between examined intersection types are not significant (they vary from 0 to 0.4 dB(A)), which can be seen from Figure 4. Previous studies [6] based on microscopic traffic simulations on a large number of simple intersection scenarios led to similar results: the intersection type was found to have a large influence on travel times, but only a small influence on global noise emission. Other studies [7] based on evaluation of noise conditions at intersections before and after conversion into a roundabout, including comparison of noise situation at regular intersection before reconstruction with activated and deactivated traffic lights, demonstrate that intersection with traffic lights has higher noise levels than roundabout (1.2 dB(A)) and intersection with traffic lights turned off (2 dB(A)).

Despite small differences in traffic noise levels given by noise modeling procedure in this research, it can be noticed that certain difference in noise emissions between examined intersection types does exist. Noise levels were highest at regular intersection with traffic lights (measuring points MM4, MM5 and MM6) while noise levels at intersection without traffic lights and mini roundabout were very similar – they vary from 0 to 0.1 dB(A) without any specific traceability.

The major advantage of roundabouts and intersections without traffic lights, with regard to decreasing traffic noise levels in the vicinity of crossroads, is a speed reduction and the fact that vehicles do not have to stop and wait under the traffic lights. In places near crossings regulated by traffic lights it is known that decelerating and accelerating traffic causes higher noise levels than free-flowing traffic because of higher engine noise levels. From aspect of intersection safety and traffic capacity, roundabouts are certainly more suitable for application in residential areas (compared to intersection without traffic lights), which is also one of the reasons why they became one of the most popular choices for intersections in urban areas. A noise problem that can occur in roundabouts is related to the application of overrun areas with track aprons at the centre in order to allow heavy vehicles to pass the roundabout. If cars use these areas to drive through the roundabouts at high speed, this may generate high impulsive-like noise levels, which may additionally increase the overall noise levels.

3 Conclusion

Considering the fact that traffic noise became one of the most invasive types of noise pollution in residential areas and that traffic conditions at road intersections can lead to additional noise pollution compared to traffic at open road segments, it was necessary to examine if noise levels in the vicinity of road intersections can be reduced. The main goal of the research presented in this paper was to establish whether intersection design has any influence on noise emissions and in accordance with that to determine which intersection type is most suitable for application in urban areas.

Noise calculation results showed that there are no significant differences in predicted noise levels between three testing models. It was noticed that intersection with traffic lights produced highest noise levels. Despite the small differences in calculated noise levels on analysed models, reductions at roundabout and intersection without traffic lights compared to intersections with traffic lights are likely to depend upon the traffic flow conditions and the intersection layout. In accordance with that, further investigations on larger number of intersection models with different input parameters (other noise emission parameters, various intersection designs) are needed.

References

[1] Jahandar, N., Hosseinpour, A., Sahraei, M. A.: Traffic Noise under Stop and Go Conditions in Intersections – A Case Study, World Academy of Science, Engineering and Technology, Vol:62., 2012.

[2] Chevallier, E., Can, A., Nadji, M., Leclercq, L.: Improving noise assessment at intersections by modeling traffic dynamics, Transportation Research Part D 14, pp. 100–110, 2009.

[3] RLS 90, Richtlinien für den Laermschutz an Strassen, vd Nr. B6510 Ver: 03/2002. Der Bundesminister für Verkehr, Abteilung Strassenbau, 2002.

[4] Ahac, S., Dragčević, V.: Road traffic noise modeling at roundabouts, Road and Rail Infrastructure II, Proceedings of the Conference CETRA 2012, Lakušić, S. (Ed.)., University of Zagreb, Faculty of Civil Engineering, Department of Transportation, pp. 751-756, 2012.

[5] Regulation on the noise mapping and content of noise maps and action plans and the calculation of permitted noise indicators, Official Journal, NN 75/2009.

[6] Coensel, B., Vanhove, F., Logghe S., Wilmink I., Botteldooren, D.: Noise emission corrections at intersections based on microscopic traffic simulation, Euronoise 2006, Finland, 2006.

[7] Ressel, W.: Evaluation of noise conditions at intersections before and after conversion into a roundabout – comparison based on empirical measurement and analytical calculation, Institut für Straßen- und Verkehrswesen Lehrstuhl für Straßenplanung und Straßenbau, 2007.

PERFORMANCE CHECKS AS PREREQUISITES FOR ENVIRONMENTAL BENEFITS OF ROUNDABOUTS

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Abstract

Environmental benefits of roundabouts are well known: roundabouts help reduce fuel consumption and vehicle emissions and also require less maintenance than traditional intersections with traffic signals, providing a reduction in energy use and costs. The prerequisite for these assets is smooth speed profile through the roundabout, which is achieved by ensuring the clarity of the situation for approaching drivers, visibility between road users, comprehensibility of traffic operations and appropriate accommodation of the design vehicles. These requirements are met through quality roundabout design. Roundabout design is an iterative process that consists of identification of initial design elements, performance checks (definition of fastest path, offtracking geometry and visibility tests) and final design details. The development of reliable software for vehicle movement simulations simplified these performance checks, and led to the exploration of new possibilities for roundabout design, which represents a logical step in the area of road traffic engineering research.
In Croatia the positive aspects of roundabouts are often annulled due to their inappropriate design. Namely, performance checks that should ensure proper speed profile and required intersection visibility are usually ignored. The main reason for this stems from the fact that there are no official national guidelines or regulations for the roundabout design. In this paper key elements for successful roundabout design will be shown, based on performance checks conducted by vehicle movement simulation software. Proposed instructions could be used for the development of national guidelines for roundabout design.

Keywords: roundabout design, offtracking control, fastest path, visibility

1 Introduction

In the past two decades roundabouts became one of the most popular choices for intersections in suburban and urban areas in Croatia: according to the available data, in the year 2008 there were more than 130 roundabouts, 85 of which were located in urban and suburban areas [1]. Their application can result in a number of benefits, which include not only the improved intersection capacity and safety, but also lower maintenance costs and less air pollution. Numerous studies showed that application of modern roundabouts can help reduce the excessive emissions and fuel consumption associated with idling time, acceleration and deceleration of vehicles that usually occur on traditional intersections with traffic signals, as well as noise pollution in the vicinity of road intersections Reported average reductions varied from 21 to 42% in emissions of carbon monoxide, 16-59% in emissions of carbon dioxide, 20–48% in emissions of oxides of nitrogen, 18–65% in emissions of hydrocarbons and 4 dB(A) in noise emission [2, 3].

Environmental benefits of modern roundabouts are directly linked to the quality of their design. Namely, in order to achieve the required reduction in vehicular emissions, the speed profile through the roundabout must be as smooth as possible [4, 5, 6] This can be ensured in the roundabout's designing phase by conducting performance checks that include offtracking geometry, definition of fastest path, and visibility tests.
Unfortunately, in Croatia the positive environmental aspects of roundabouts are often annulled due to their inappropriate design. Namely, performance checks that should ensure proper speed profile and required intersection visibility are usually overlooked. The main reason for this stems from the fact that there are no official national regulations for the roundabout design. Because of that, in this paper focus is set to display the key elements of roundabout performance checks described in several design guidelines that are applied in countries with lasting tradition of roundabout design and construction.

2 Roundabout performance checks

Roundabout design is an iterative process that consists of identification of initial design elements, performance checks (which include definition of fastest path, offtracking geometry and visibility tests), and final design details. Roundabout design in Croatia is, due to the lack of official guidelines, usually carried out according to the guidelines of the Institute of Transport and Communications [7], German guidelines [8], Austrian guidelines [9] and/or Swiss norms [10]. These guidelines and norms do not include detailed instructions for all performance checks, particularly for the speed checks, i.e. definition of fastest path, which is essential for safe and functional roundabout design. Because of that, the key elements of roundabout performance checks represented in documents that are not commonly referred to by Croatian designers, namely FHWA's Roundabouts: An Informational Guide [11], and Serbian regulations on road and intersection design [12], are also described.

Figure 1 Offtracking control at roundabouts

2.1 Offtracking control

In order to secure safe and unobstructed traffic flow on any type of intersection, it is crucial to investigate the design vehicle's ability of passing through the intersection. This in ensured by means of offtracking control. In accordance with [8, 9, 10, 12], a drivable roundabout offtracking control should meet the following requirements:

- design vehicle selected for this performance check must correspond to the intersection position in the road network,
- offtracking control must be conducted for all approaches on roundabout and for all three directions (straight, right and left, as shown in Figure 1.),
- the safety lateral width of at least 0.25 m must be ensured along all elevated curbs.

In the past, offtracking control was usually conducted by means of design vehicle's templates. Issues concerning the usability of these templates for the said performance check on modern roundabouts include following limitations. Templates are given for limited number of design vehicles, turning radii, scales and steering angles and can be applied only for offtracking control on right turns. Because of that, offtracking control on modern roundabouts should be conducted by means of verified swept path analysis software.

2.2 Speed analysis

Another key performance check of a roundabout concept is speed analysis. According to [11, 12], this analysis is composed from following steps:
- construction of the fastest path through the roundabout for each approach,
- definition of five critical path radii for each approach (Figure 2) and
- vehicle speed estimation based on speed-radius relationship (Figure 3).

Fastest path is defined as the smoothest, flattest path possible for a single vehicle, in the absence of other traffic and ignoring all lane markings. The fastest path is drawn for a vehicle traversing through the entry, around the central island, and out the relevant exit [11, 12]. It can be drawn by hand, or constructed by means of CAD software, using 3rd degree splines (piecewise polynomials of 3rd degree with function values and 2nd derivatives that agree at the points where they join). For the purpose of speed analysis on roundabouts, fastest path is drawn with safety lateral width of 1.5 m along all elevated curbs.

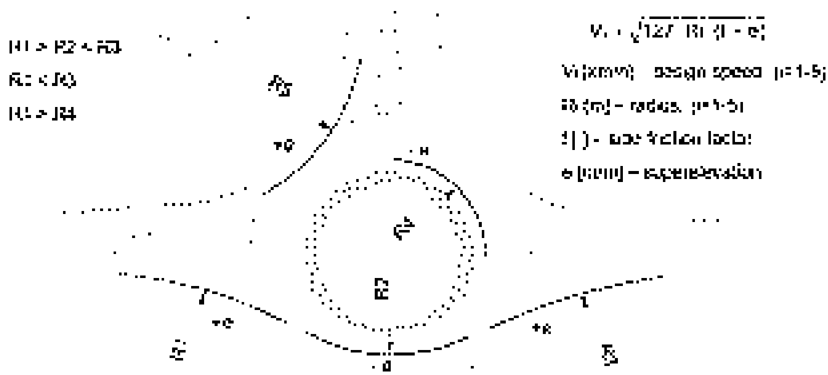

Figure 2 Fastest path and five critical path radii

After sketching the fastest path, the minimum radii are measured using suitable curve templates or by replicating the path in CAD and using it to determine the radii. Vehicle speed estimation is based on speed-radius relationship, which is described in [11, 12 13] and shown on Figure 2.
In order to achieve required entry (V1, V5), circulating (V2, V4), and exit speeds (V3), appropriate critical radii relationships for each approach must be achieved. Entry path radius (R1)

must be larger than circulating path radii (R2 and R4), but at the same time smaller than exit path radius (R3); also, circulating path radius (R2) must be smaller than exit path radius (R3). In addition to appropriate (maximum) speeds, speed checks should consider the speed differential between conflicting traffic movements. According to [11], in order for vehicles to safely negotiate the roundabout, the maximum speed differential between movements should be approximately 15 to 25 km/h, while according to [12] this value should be approximately 10 to 20 km/h. The predicted speed values are used to assess a roundabout's safety performance and are used in the intersection and stopping sight distance computations.

2.3 Visibility

An important aspect of roundabout design is the provision of adequate sight distance for all approaches. To safely negotiate the roundabout the driver must clearly perceive and understand the permitted manoeuvres before reaching the intersection – drivers approaching the roundabout should be able to recognize the main canalization features of the roundabout (the central island, raised splitter islands) and slow down to the appropriate speed [14].

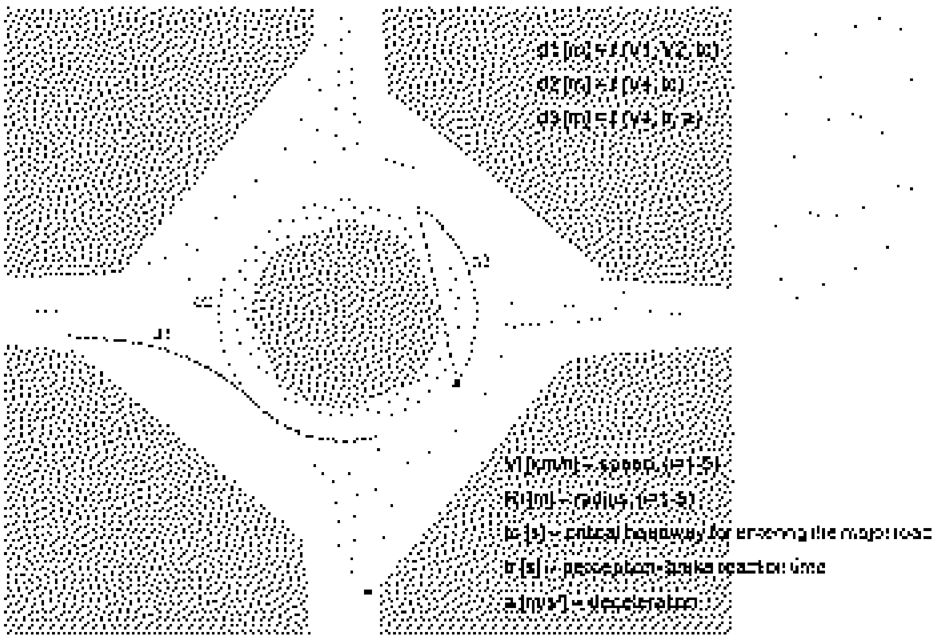

Figure 3 Fields of vision on roundabouts

In order to provide the adequate sight distance, visibility checks (that include investigation of stopping sight distance on every approach and on circulatory roadway, and also intersection sight distance for each approach) should be conducted during the roundabout design. Required sight distances on the approaches are directly related to predicted speed values on fastest paths through the analysed roundabout (V1, V2 and V4), critical headway for entering the major road (tc), while sight distance on circulatory roadway is related to predicted speed on circulatory roadway (V4), perception-brake reaction time (tr) and deceleration (a) [11, 13]. Visibility checks result in sight lines that should be overlaid onto a single drawing, as shown in Figure 3. This graphic presentation of fields of vision on roundabouts can be used as guidance on the appropriate locations for various types of visual obstructions. Namely, excessive intersection sight distance can lead to higher speeds that reduce intersection safety, so in order to restrict sight distance to the (recommended) minimum landscaping or other visual obstructions are used [11].

The white zones in Figure 3 are areas that should be clear of large obstructions that may impede driver's line of sight. In the areas with grey solid shading, especially within the central island, visual obstructions may be used to break the forward view for through vehicles, which should contribute to speed reductions and reduction of oncoming headlight glare [11].

3 Speed profiles on roundabouts

Numerous studies have shown that all roundabouts have an influence zone over which they exert a significant speed effect, forcing the driver to decelerate from cruise (free flow) speed to circulating speed and to accelerate back to cruise speed when leaving the intersection. This influence zone usually extends from the centre of the intersection to each intersection leg for about 80 to 100 m in total [15, 16].

Based on the extensive empirical measurements, three distinct speed profiles for a vehicle moving through a single lane roundabout were identified, which cover all combinations of traffic flow conditions [4]:

- an unstopped vehicle that approaches the roundabout, decelerates to negotiate the roundabout circular roadway and then accelerates as it exits the roundabout,
- a vehicle experiencing a single stop, in which there is a full deceleration to a stop at the yield line, while waiting for an acceptable gap in the circulating lane, then accelerating to the circulating and cruise speeds, and
- a vehicle that experiences multiple stops on the approach as it moves up the queue.

It was also found that high speed differential between circulating and cruising speed will result in excessive accelerations and decelerations, with accompanying inconsistent driver behaviour, increase in emissions and fuel consumption [4, 16].

It was established that the emissions increase in following situations:

- when queuing conditions prevail as conflicting traffic increases,
- at low conflicting traffic volumes, when the acceleration rate back to cruise speed is high, and
- at larger differences between the cruise and circulating speeds [4].

Critical sections on a single lane roundabout in terms of their relative impact on total emissions is the section on which vehicles experience short stop and go cycles (in front of circulating traffic yield line) and the section on which vehicles accelerate back to cruise speed [4]. Because of that, it is important to ensure the appropriate speed differential between consecutive geometric elements through quality roundabout design. As mentioned earlier, speed checks should ensure that relative speeds between conflicting traffic streams and between consecutive geometric elements are minimized to 20 (25) km/h. Speed checks must also ensure that relationships between predicted speeds on fastest paths through the roundabout are as follows:

- entry speed (V1) must be smaller than exiting speed (V3),
- circulating speed (V2, V4) must be smaller than entry speed (V1),
- maximum entry speed (V1, V5) should be smaller or equal to 40 km/h.

4 Conclusions

Environmental benefits of modern roundabouts are directly linked to the quality of their design. In order to achieve the required reduction in vehicular emissions, the speed profile through the roundabout must be as smooth as possible, which can be ensured in the roundabout's designing phase by conducting performance checks (analysis of offtracking geometry, definition of fastest path, and visibility tests).

In Croatia the positive environmental aspects of roundabouts are often annulled due to their inappropriate design: performance checks that should ensure proper speed profile and required intersection visibility are usually overlooked. The main reasons for that are the facts that there are no official national regulations for the roundabout design, and that design is usually carried out according to the guidelines and/or norms which do not include instructions for all required performance checks.
Apart from roundabout design guidelines and norms that are usually referred to by Croatian designers, future official national regulations on roundabout design should include performance checks described in FHWA's Roundabouts: An Informational Guide and/or Serbian regulations on road and intersection design, which are presented in this paper. Also, in order to ensure the required environmental benefits of roundabouts, described performance checks should result in smooth speed profiles and minimized relative speeds between conflicting traffic streams and between consecutive geometric elements (to maximum 25 km/h).

References

[1] Legac, I., Pilko, H. & Šubić, N.: Introduction on roundabouts in Croatia – preliminary experiences, 16th IRF World Congress, Lisboa, 2010

[2] Hydén, C., Várhelyi, A.: The effects on safety, time consumption and environment of large scale use of roundabouts in an urban area: a case study, Accident Analysis and Pre6ention 32, 11–23, 2000

[3] Mandavilli, S., Russell, E.R., Rys, M.J.: Impact of Modern Roundabouts on Vehicular Emissions, Proceedings of the 2003 Mid-Continent Transportation Research Symposium, Ames, Iowa, USA, 2003

[4] Coelho, M.C., Farias, T.L., Rouphail, N.M.: Effect of roundabout operations on pollutant emissions, Transportation Research Part D: Transport and Environment, Volume 11, Issue 5, 2006

[5] Chamberlin, R., Swanson, B., Talbot, E., Dumont, J., Pesci S.: Analysis of MOVES and CMEM for evaluating the emissions impacts of an intersection control change, paper presented at TRB 90th Annual Meeting: Current Environmental Issues in Transportation, Washington, D.C., USA, 2011

[6] Ahn, K., Kronprasert, N., Rakha, H.: Energy and Environmental Assessment of High-Speed Roundabouts. Transportation Research Record No. 2123

[7] Dadić, I., Tollazzi, T., Legac, I., Čičak, M., Marić, V., Kos, G., Brlek, P.: Smjernice za projektiranje i opremanje raskrižja kružnog oblika – rotora, Institut prometa i veza, Zagreb, 2002

[8] Merkblatt für die Anlage von Kreisverkehren, K 10000, FGSV (Forschungsgesellschaft für Strassen- und Verkehrswesen), Köln, 2006

[9] Plangleiche Knoten – Kreisverkehre, Österreichische Forschungsgesellschaft Straße-Schiene-Verkehr (FSV), (RVS 03.05.14), Wien, 2010

[10] SN 640 263 – Schweizer Norm, Knoten mit Kreisverkehr, VSS, Zürich, 2000

[11] ROUNDABOUTS: An Informational Guide, Second Edition, Transportation research Board, Washington, D.C., USA, 2010

[12] Pravilnik o uslovima koje sa aspekta bezbednosti saobraćaja moraju da ispunjavaju putni objekti i drugi elementi javnog puta, Službeni glasnik RS, br. 50/2011

[13] AASHTO Green Book – A Policy on Geometric Design of Highways and Streets, AASHTO, Washington, D.C., USA, 2004

[14] Pratelli, A., Souleyrette, R. R., Harding, C.: Roundabout perception: review of standards and guidelines for advanced warning, WIT Transactions on The Built Environment, Vol 111, WIT Press, 2010

[15] Taylor, M.A.P. , Rutherford, L.M.: Speed profiles at slow points on residential streets, Proceedings of the 13th ARRB/5th REAAA Conference, Vermont South, Victoria, Australia, 1986

[16] Brindle, R.E.: Speed-Based Design of Traffic Calming Schemes, Proceedings of the ITE 2005 Annual Meeting and Exhibit Compendium of Technical Papers, Melbourne, Australia, 2005

URBAN PAVEMENT SURFACES HEATING – INFLUENCING PARAMETERS

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Abstract

Urban heat islands are among most serious environmental problems in cities because overheating of the pavement surfaces during summer season causes discomfort for citizens and increased consumption of energy. The strategies for solving this problem are different but using materials which have favorable thermal characteristics is emphasized as important part of those strategies. Understanding of parameters that affect and contribute to the urban heat island and more precisely to the increased heating of the urban pavement surfaces is necessary in that process to assure that future reconstructions and constructions in urban areas are done in the best possible way.
Research done in the field showed that urban heat islands appear in areas where wind velocity and evapotranspiration are decreased (less green areas) and level of waste heat is greater.
Field research in the city center of Rijeka and on the pilot field at the location of the University Campus (outside center in less built area) was done during summer season of 2011, 2012. and 2013. The task of the field research was primarily to identify local materials which use can help reduce or mitigate the effect of additional heating in the urban environment caused by emission of heat from pavement surfaces but also to analyze the impact of other parameters (humidity, wind speed, presence of water, traffic intensity) on the heating of urban pavement surfaces. In the paper short overview of the parameters influencing heating of urban pavement surfaces will be done and results of the temperature measuring on cross section situated in the city centre

Keywords: pavement surfaces, pavement materials, urban areas, urban heat island, temperature

1 Introduction

Modern urban areas have typically darker surfaces and less vegetation than their surroundings. These differences affect climate, energy use and habitability of cities [1]. Materials commonly used in urban areas, such as asphalt and concrete, have significantly different thermal and optical properties than the surrounding rural areas. As vegetation and natural surfaces are replaced by asphalt and concrete surfaces for roads, buildings and other structures necessary to accommodate growing population, heat islands are developed; urban materials alter energy balance of an urban surface as they absorb, rather than reflect, the incoming solar radiation causing surface temperatures and overall ambient temperatures to rise. Lack of vegetation in urban areas also affects the energy balance, as the natural cooling of the surface by evapotranspiration is minimised [2]. The phenomenon of urban heat island (UHI) is related to positive thermal balance created in the urban environment because of the increased heat gains like the high absorption of solar radiation and the anthropogenic heat, and

the decreased thermal losses. Uncontestably, UHI is the most documented phenomenon of climate change and is very well documented for various geographic areas of the planet [3,4]. The impact of pavements on the development of UHI is very important. Many recent studies have shown that paving surfaces play a very determinant role on the overall urban thermal balance [5,6]. Pavements cover a quite high percentage of the urban fabric and contribute highly to the development of UHI. Paved surfaces in Europe and USA, consist mainly of asphalt and concrete surfaces that present high surface temperatures during the summer period [4, 7].
The aim of this paper is to present part of the results of temperature measurements on different types of pavement surfaces. Measurements were conducted in the City of Rijeka (Croatia) and are first known to the authors measurements of this kind conducted in Croatia. Measurements were conducted during the summer period when the temperature of road surface is reaching its peak. Analysis of the results will be used to comment possible parameters that can contribute to increase or to the decrease the temperature of urban pavement surfaces.

2 Pavements and urban climate

As was stated in the introduction, pavements affect strongly the urban climate. Their thermal balance is determined by the amount of the absorbed solar radiation, the emitted infrared radiation, the heat transferred by convection to the atmospheric air, the heat stored into the mass of the material and the heat conducted to the ground. The thermal balance of pavements and its impact on urban climate is analysed using experimental and computational simulation techniques. Microscale measurement techniques provide detailed information on the thermal conditions of the studied areas and allow a deep understanding of the corresponding thermal processes. Most of the microscale experimental studies have been performed using infrared thermography or conventional temperature measurements using thermocouples. The use of thermocouples to measure surface temperatures is a well known and accurate technique provided that a very good contact is achieved between the surface and sensors [4].

2.1 The role of the parameters that affect pavement heating

As was stated in the introduction, urban areas typically have surface materials which have a lower albedo than those in rural areas. As a result, the built environment generally reflects less and absorbs more of the Sun`s energy. This absorbed heat increases surface temperatures and contributes to the formation of urban heat islands. The value of the reflectivity is determined by the colour of the material and its roughness. Light colours present a lower absorptivity to the visual spectrum of solar radiation, while the specific absorptivity to the infrared part of the radiation is quite independent of the perceived colour [3, 4]. Numerous studies have been performed to correlate the impact of the colour of pavement materials on their surface temperature and sensible heat release. The research, conducted by Doulos et all [8] in which behaviour of 93 samples of commercial pavement materials were analysed under the same insolating conditions, has pointed out those analyzed materials which, under the determined micro-climate conditions (Athens, Greece) can contribute to reducing the UHI, reducing the electricity consumption and improving thermal conditions in open areas.
Materials emit long wave radiation as a function of their temperature and emissivity. High emissivity values correspond to good emitters of long wave radiation and can readily release the absorbed energy [3, 9]. Several studies were performed to understand the impact of the emissivity on the thermal performance of materials used in the urban environment. The research, conducted by Syneffa et all [9] showed that emissivity is the most important factor affecting the surface temperature of the materials during the night period.
Besides these properties, the thermal behaviour of urban surfaces plays an important role in the UHI development. This behaviour is largely determined by the density, specific heat

capacity, thermal conductivity and thermal admittance coefficients of the used materials. Heat capacity refers to ability of material to store heat, while thermal conductivity is ability of material to conduct heat. Both entities determine thermal diffusivity of material, which is an important indicator of how easily heat can penetrate into material. The ability of material to store and release heat is defined by the thermal admittance which is a product of thermal conductivity and heat capacity. Increased thermal conductivity of paving surfaces contributes to transfer faster the heat from the pavements to the ground and vice versa [4]. The research, conducted by Gavin et all [10] have showed that with increasing conductivity the average maximum surface temperature decreases, while average minimum temperature increases. In the same study the same correlation is showed for thermal capacity.
Water also plays a very important role. In permeable pavements, water passes to the soil. It evaporates when the temperature of the material increases and thus lowers a pavement surface temperature. Evaporation depends on moisture content in the material and atmosphere and depends of materials temperature [4, 11].
Use of high-albedo urban surfaces and planting of urban trees are inexpensive measures that can reduce summertime temperatures. When trees are planted and albedo is modified throughout an entire city, the energy balance of the whole city is modified, producing city-wide changes in climate. Phenomena associated with city-wide changes in climate are referred to as indirect effects [1].

3 Pavement surface measurements in the City of Rijeka

3.1 Overview of the published results and findings

An extensive study of the pavement heating was conducted during summer seasons 2011., 2012. and 2013. in the City of Rijeka. Part of the research was done in the city centre where temperature of surfaces made of different materials (asphalt, concrete, stone, granite) were compared and during 2013. data were collected at the pilot field at the pavement test sections at the location of the Campus of the University of Rijeka (located outside of the centre) at the higher elevation (150-180 masl). In this paper part of the results of the study conducted in the city centre will be presented.
Interesting analyses of the influence of different parameters on the pavement surface temperature can be done if considered a cross section in urban area. At the Picture 1 the cross section defined in the Centre of Rijeka City is shown together with the details about surface materials. In this case the cross section is situated partly in the intensively built pedestrian zone, partly on canal and partly on the big parking area. Few streets with high traffic volume are taken into analyses too and also pedestrian bridge made of aluminium and glass. At the location the temperature was measured during 3rd and 4th July 2012. during 24 hours in period of 30 minutes.
The results [12, 13, 14] showed that asphalt is certainly, regarding thermal properties, the most unfavourable material to be used in urban areas. Differences have been noticed if lighter aggregate (limestone) is used or if asphalt pavement is in use for some time and has washed surface. Concrete can be estimated as more favourable than asphalt for urban pavements because exposed to the same temperature conditions develops lower surface temperatures and consequently realises less heating to the surrounding air.
The chart from Figure 1 shows changes in the air temperature and surface pavement temperature measured during July 2012 at the locations near cross section described and analysed in paragraph 3.2. From the picture is evident that asphalt surface has highest surface temperature compared with stone and concrete placed in the same conditions [15].
Except the type of material used for pavements the Study pointed out some other parameters important for formation of urban heat island in the cities: position of the surface inside urban area, presence of water and exposure to isolation are among most important.

Figure 1 Cross section – location and materials

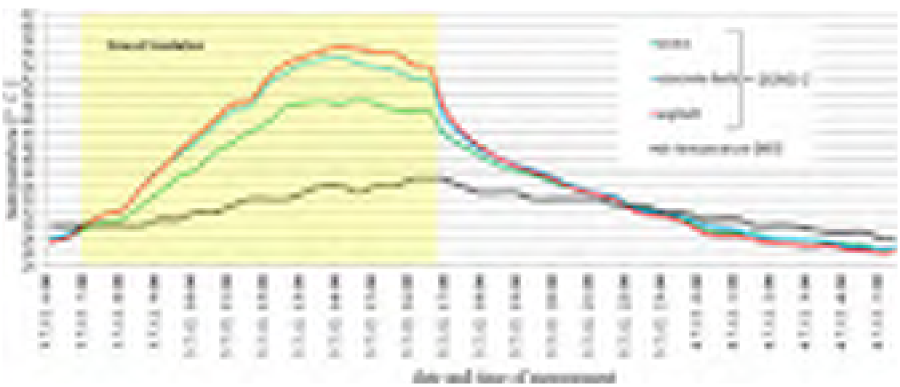

Figure 2 Whole day temperature measurement on different pavement surfaces on 3rd July 2012. [15]

3.2 Cross section in the City centre of Rijeka

The temperatures of the air and pavement surfaces measured at 7 a.m. and at 1 p.m. at the cross section (Figure 1) are showed at the chart (Figure 3). It is evident that the difference in air temperature and surface temperature in the morning period is significantly lower than in afternoon when the pavement at specific locations is more than 10°C warmer than air.
The results are showed in the way that temperature of different sub-areas can be compared and sub-areas are showed both with pictures and symbols.

The influence of the presence of water can be established as the factor that decreases temperature of the air as well as the temperature of the surfaces around the water, in this case the canal. Even being very small (positions 17-19) the green zone with some trees has positive effect on the pavement temperature nearby and they are for few degrees lower than on the pavement open to the insolation.
Temperature of the closed and intensively build area (square surrounded with houses) is slightly lower than temperature measured on the open and exposed area of the parking. The conclusion can be that it has to do with the type of surfaces as well as with the duration of exposure to the direct insolation. Pavements in pedestrian zone are made of stones and that kind of pavements show better characteristics regarding heating than asphalt pavement (see Figure 1) [12, 13].
Slightly higher temperatures were also measured at the position of traffic lines on the roads (15-16, 28-29) and it can be presumed that the influence of intensive traffic is negative, e.g. that intensive traffic volumes can contribute to the increase of pavement temperature, even not very much.

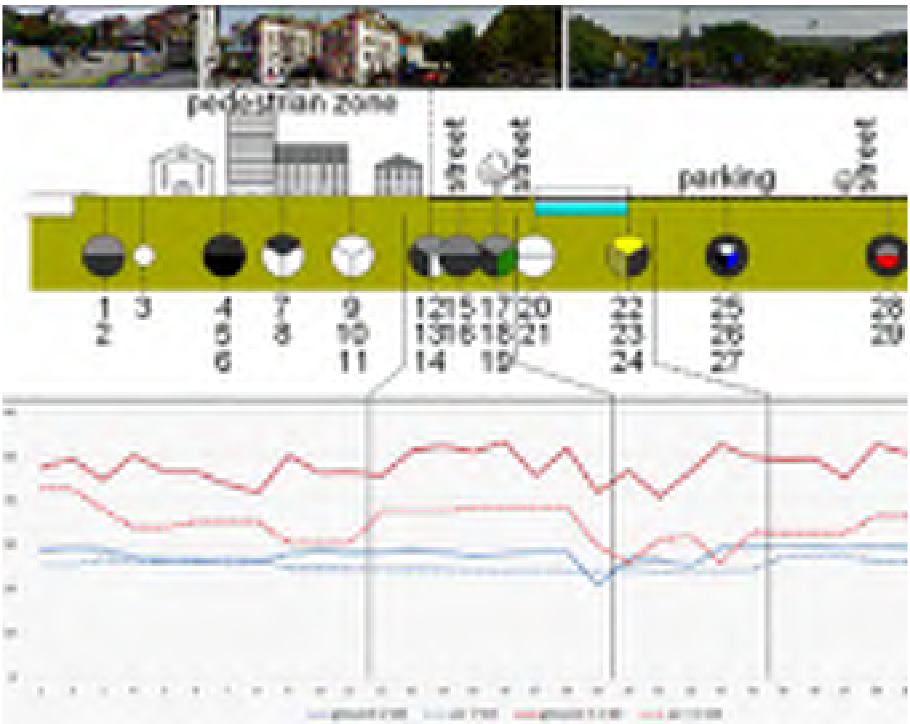

Figure 3 Cross section – temperatures of the air and pavement

4 Conclusion

It is known that pavements surfaces play a very determinant role on the overall urban thermal balance. Different are the parameters that influence that relation, some are connected with the material properties and some with the location of the urban area. Previous analyses done by the authors pointed out which of the materials have positive influence on UHI. From data collected at the field study of pavement temperatures in the City of Rijeka (Croatia) in this paper a temperature of one central area is shown. Since the cross section included

areas within pedestrian zone and the zone of densely built areas as well as areas near canal and large parking areas, it was possible to analyse environmental conditions that affect the temperature. It turned out that more protected areas within the built-up area (so called urban canyons) that are not exposed to constant sunlight, the areas near water and areas within greenery, have lower average temperature than parking surfaces and traffic lanes that are exposed to traffic and direct sunlight.
Further analyses that will include calculation of the average temperature in the zone built of different materials as well as data about solar radiation must be done. In that way, more precise conclusions can be done.

References

[1] Akbari, H., Pomerantz, M. & Taha, H.: Cool surfaces and shade trees to reduce energy use and improve air quality in urban areas, Solar Energy, 70(3), pp. 295-310, 2001.

[2] Stathopoulou, M., Synnefa, A., Cartalis, C., Santamouris, M., Karlessi, T. & Akbari, H.: A surface heat island study of Athens using high-resolution satellite imagery and measurements of the optical and thermal properties of commonly used building and paving materials, International Journal of Sustainable Energy, 28(1-3), pp. 59-76, 2009.

[3] Santamouris, M.: Heat island research in Europe: The State of the Art, Advances in Building Energy Research, 1, pp. 123-150, 2007.

[4] Santamouris, M.: Using cool pavements as a mitigation strategy to fight urban heat island-A review of the actual developments, Renewable and Sustainable Energy Reviews, 26, pp. 224-240, 2013.

[5] Sailor, D.: Simulated urban climate response to modifications in surface albedo and vegetative cover, Journal of Applied Meteorology, 34, pp. 1694-1700, 1995.

[6] Surabi, M., Akbari, H., Mahanama, S., Sednev, I. & Levinson, R.: Radiative forcing and temperature response to changes in urban albedos and associated CO2 offsets, Environmental Research Letters, 5, 014005, 2010.

[7] Tran, N., Powel, B., Marks, H., West, R. & Kvasnak, A.: Strategies for design and construction of high-reflectance asphalt pavements, Journal of the Transportation Research Board 2098, pp. 124-130, 2009.

[8] Dolous, L., Santamouris, M. & Livada, I.: Passive cooling of outdoor urban spaces. The role of materials, Solar Energy, 77, pp. 231-249, 2004.

[9] Syneffa, A., Santamouris, M. & Livada, I.: A study of the thermal performance of reflective coatings for the urban environment, Solar Energy, 80(8), pp. 968-981, 2006.

[10] Gavin, G.J., Phelan, P.E., Kaloush, K.E. & Golden, J.: Impact of pavement thermophysical properties on surface temperatures, Journal of Materials in Civil Engineering, 19(8), pp. 683-690, 2007.

[11] Cuculić, M. & Šurdonja, S.: Pregled novijih kolničkih konstrukcija koje ublažavaju efekt urbanih toplinskih otoka, Zbornik radova. knjiga XIII, pp. 245-262, 2010.

[12] Babić, S., Deluka-Tibljaš, A., Cuculić, M. & Šurdonja, S.: Analiza zagrijavanja kolničkih površina urbanih područja,Građevinar 64, (2); pp. 125-132, 2012.

[13] Cuculić, M., Babić, S., Deluka-Tibljaš, A. & Šurdonja, S.: Pavement surfaces in urban areas, Road and rail infrastructure II, Proceedings of the 2nd international conference on road and rail infrastructure – CETRA 2012, 7–9 May 2012, Dubrovnik, Croatia, pp. 273-279.

[14] Cuculić, M., Šurdonja, S., Babić, S. & Deluka-Tibljaš, A.: Possible locations for cool pavement use in city of Rijeka, The 4th EPAM, European pavement and asset management, Malmo, poster presentation, 2012.

[15] Deluka-Tibljaš, A., Šurdonja, S., Babić, S. & Cuculić, M.: Analyses of urban Pavement surface temperatures, The Baltic Journal of Road and Bridge Engineering (in press).

BURIED FLEXIBLE CORRUGATED STEEL STRUCTURES – MODERN TECHNOLOGY IN CONSTRUCTION OF WILDLIFE CROSSINGS

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Abstract

Even though modern road network is the main prerequisite for sustainable economic development of every country, intensive road construction also has a negative influence on surrounding life and environment. With its unique characteristics, motorways presents large obstacle in landscape, intersecting the habitual migratory wildlife routes in their natural surroundings. One of the measures from adverse motorway impacts includes ensuring natural wildlife migrations across motorways. One of such measurements is construction of wildlife crossings which are well-established in the Croatian road planning process and relevant legislation. Therefore, it is created a favourable environment for the development and adoption of new technologies for their construction.
Buried Flexible Corrugated Steel Structures (BFCSS) presents modern construction technology for wildlife crossings with the variety of opportunities in term of spans and shapes. Technology of BFCSS for civil engineering infrastructure projects is well known in countries with developed steel industry and expensive labour. But in this part of Europe, south-eastern Europe, it has been unfairly neglected and just sporadically used. In order to promote new technology and achieve progress in the field of road construction, modern way of wildlife crossings construction will be presented.
In this paper, basic engineering properties so as the designing details and usage possibilities of BFCSS will be presented. Special emphasis will be given on practical experience of their use in the construction of wildlife crossings as well as on the use of long-span structures.

Keywords: Buried Flexible Corrugated Steel Structures, wildlife crossings, long-span structures, design details

1 Introduction

High-quality road network, especially the motorway network incorporated in the global transportation system is one of the basic conditions for the development of every country. At the same time, number of road users are in constant increase, so current road network is reaching even the most remote corners of the world. However, besides undisputable positive effects of road construction on global development, some negative impact have to be mentioned. Increasing built of motorways drastically affect the landscape, and it has high influence on human life and even higher on the animal and plant life. So the community attention to the environmental aspects of road construction is increasing.
There is a variety of measures implemented in road built for environmental protection. Some of protective measures from adverse road built are noise protection, drainage system, lan-

dscaping of the road area, protection of archaeological finds and measures for ensuring natural migration of animals across roads.
Habitat fragmentation has been recognized as one of the most significant factors contributing to the decline of biodiversity in Europe [1]. So, with its unique characteristics, motorways presents large obstacle in landscape, intersecting the habitual migratory wildlife routes in their natural surroundings. For that reason, it is very important to devote adequate attention during the design development stage for new motorway projects to provide suitable wild animals passageways. For that purpose, four types of structures have been conceived: green bridges, wildlife crossings and passageways, pipes and ground channels for small mammals and other vertebrates and amphibian tunnels [2].
Wildlife crossings are constructions that allow roads permeability for animals and safe crossing of wild animals at appropriate spatial intervals. By the Regulations on wildlife crossings [3], the construction of these structures is the obligation of roads investors and this specially (dedicated) built crossings have protection as natural values.
Construction of wildlife crossings are well-established in Croatian road planning process and relevant legislation. There are 80 wildlife crossings on 1250,7 km of motorways and semi-motorways in Croatia, which means one crossing on nearly every 16 km. One of them is presented in Fig. 1.

Figure 1 Wildlife crossing on Croatian motorway [2]

Therefore, it is created a favourable environment for the development and adoption of new technologies for their construction. Buried Flexible Corrugated Steel Structures (BFCSS) presents modern construction technology for wildlife crossings with the variety of opportunities in term of spans and shapes. So, basic engineering properties, designing details and usage possibilities of BFCSS are following.

2 Technology of Buried Flexible Corrugated Steel Structures

Animal passages are built as underpasses and overpasses. They can have various shapes and can be built in various technologies.
Since the introduction of BFCSS in XIX century this technology was gradually used in road construction. At the very beginning, there was galvanized steel pipes with round cross section, now referred to as corrugated steel pipes. The economy of these pipes led to a larger diameter, which are assembled on site from curved, corrugated plates. Corrugation of these plates was around 150x50mm, and they are now referred to as corrugated steel structures.
Nowadays this kind of structures are widely known and applied in many countries all over the world. They are mostly used as culverts or bridges for the roads and railway infrastructure, but they are also widely used for many other applications in the industrial, mine and other market sectors [4].
Today corrugated steel pipes are produced with corrugation of 68x13 mm, 76x25 mm, 120x20 mm, 125x26 mm and similar. Corrugated steel structures are manufactured with corrugation of 152x22 mm, 150x50 mm, 200x55mm and similar (fig. 2).

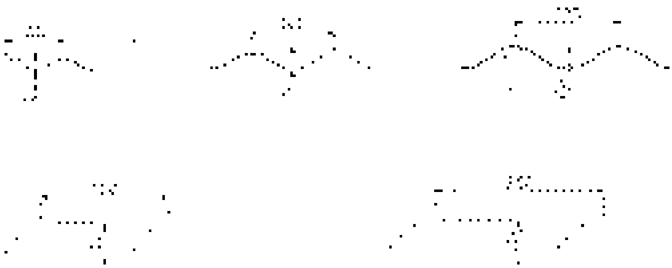

Figure 2 Typical shallow corrugation of BFCSS

Used us culverts or bridges under the road, maximum span that can be achieved with corrugation of 150x50 mm or 200x55 mm is about 12 m. They can be used with many different shapes showed in figure 3. Shape called box culvert (fig. 3i) can have a maximum span of 7,00 m.

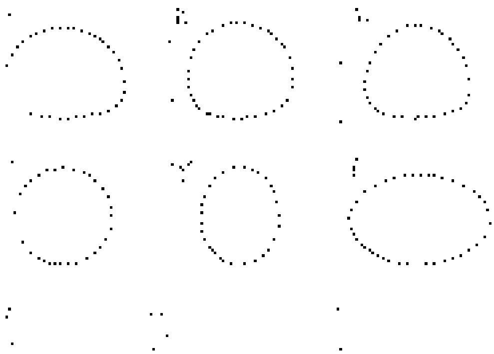

Figure 3 The most popular shapes of BFCSS

In the mid of 80's of the twentieth century deep-corrugated plates were introduced, with a pitch between 380 and 400 mm, and rise between 140 and 150 mm (fig.4), they initiated a new generation of corrugated steel structures. This kind of structure are called SuperCor, DeepCor, BridgeCor, StrenCor.

Figure 4 Deep corrugation

The advantage of deep corrugation is that, with a small increase in the volume of steel, the flexural rigidity is increased about nine times with comparison to shallow corrugation 200x55 mm. Thanks to that it is possible to make BFCSS with greater span up to 24 m, called super-span or long-span. It was possible to achieve bigger span also because of reinforcement ribs patented in 1994.

Figure 5 Animal overpass in Trzebaw (2002) (span of 17,68m) and above railway, Rzepin (2008); Poland

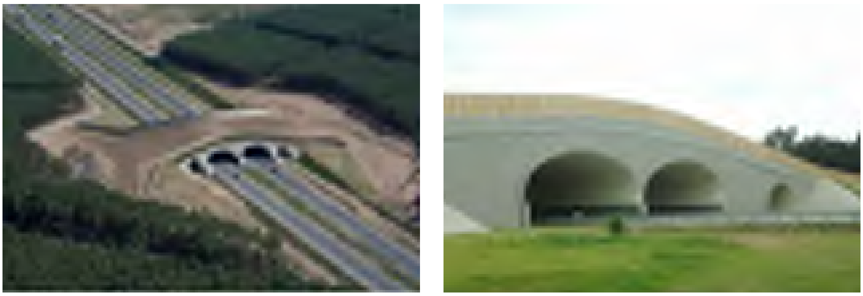

Figure 6 Animal overpass: motorway A2, Poland; constructed in 2006 and 2013.

Figure 7 Animal overpasses above highway, Banff; Canada (2009) and South Korea (2010)

Figure 8 Animal overpasses, Tallinn, Estonia 2013

A new dynamic development of BFCSS will not be possible without in situ and laboratory tests and development of design method. One of the newest design method including new innovations and trends is Sundquist–Petterson method described in [5].
New generation of deep corrugated BFCSS gives a possibility to design and install structures that can be used as bridges breaking next technological barriers for road and railway infrastructure and civil engineering. There are many of such structures already built or designed and those which are waiting for realization used like viaducts, and especially ecological animal crossings over new motorway net in the center of Europe [6].
Advantages of constructing of animal crossing with the use of BFCSS versus traditional technologies are lower cost, faster construction, minimum disruption to traffic and nice fit into the landscape. Important environmental factor is a use of soil as a structural element. Very often local soil can be used for backfilling.
BFCSS can be efficiently used to build animal crossing over roads. Quick construction and minimum disturbance to surrounding limit negative influence of works on environment and save cost due to minimum disturbance of the traffic. Use of natural materials (sand, gravel, stone, wood) enhances environmental effect. Financial effectiveness and aesthetic look support the idea of common use of those technology for construction of animal crossing over and under roads. Figures 5- 8 show examples of such animal overpasses.

3 Croatian experience with Buried Flexible Corrugated Steel Structures

Some of the first experiences with corrugated steel structures in Croatia were the use of corrugated steel pipes for the construction of culverts. In [7], one of such kind of an example is presented. In Croatia and Balkan's region BFCSS are only used for some several years, and there are structures in the road network with the use of this technology also as animal crossing.
In May 2009, animal underpass in km 5+030,00 of Adriatic Highway – ISTRIAN “Y” was constructed. It’s the passage of wild and domestic animals with the light opening passages 4x4m. To fulfill this requirement, corrugated steel structures with span 5,97m and height 5,49m was used, with corrugation 200x55 mm. Total length 46,44 m. In September 2009 another animal underpass in km 43+849,02 of Adriatic Highway – ISTRIAN “Y” was constructed by use of profile with span 4,49 m and height 3,97 m. Total length 39,32 m.
Both structures were constructed in two phases. In a first phase around 65% of the total length of underpasses were built under first highway line, and in the second phase second parts of structures were installed later. In Fig. 9 Phase II and finished animal underpass is presented.

Figure 9 Installation of animal underpass – phase II and animal underpass in km 5+030,00 of Adriatic Highway – ISTRIAN “Y”

Based on the above examples, we can say that Croatia has made the first step towards adopting a technology of BFCSS and use it for animal crossing. Beyond these examples, there are many small culverts in Croatia with use of BFCSS. There are many ready designs that are waiting for realization.

4 Conclusion

Even though, technology of BFCSS for civil engineering infrastructure projects is well known in countries with developed steel industry and expensive labour, in Croatia and generally in south-eastern Europe, it has been unfairly neglected and just sporadically used. Construction of wildlife crossings are well-established in Croatian road planning process and relevant legislation so favourable environment for the development and adoption of new technologies for their construction is created. In order to promote new technology and achieve progress in the field of road construction, modern way of wildlife crossings construction is presented in this paper.

Some of the basic aadvantages of BFCSS use in construction of animal crossing are lower cost (average 30-40% cheaper than traditional solutions), faster construction (execution is several times faster as compared to concrete structures), minimum disruption to traffic and nice fit into the landscape. Also, use of natural materials (sand, gravel, stone, wood) enhances its positive environmental effect.

However, it could be concluded that so far, this technology is not fully utilized in Croatian territory. There is no deep corrugation structure built in Croatia yet.

Taking into consideration advantages given by BFCSS, especially for its application in animal crossings construction and positive experience from its usage in other European countries might result in a greater application of steel culverts in Croatian civil engineering practice.

References

[1] Huber, Đ., Kusak, J. & Vivoda, B.: Green bridges and other structures for permeability of highways in Croatia for animals, 5th Croatian Road Congress, Cavtat-Dubrovnik, 2011.

[2] Crnjak, M., Puž, G., Marić, A. & Čleković, V.: Croatian Motorways, Hrvatske autoceste, 2008.

[3] Regulations on wildlife crossings (in Croatian), NN 5/07

[4] Janusz, L. & Madaj A.: Obiekty inżynierskie z blach falistych, Projektowanie i wykonawstwo, 2009.

[5] Pettersson, L. & Sundquist, H.: Design of soil steel composite bridges, 4th edition, 2010.

[6] Bednarek, B. & Czerepak, A.: Animal crossing built over A2 motorway in Poland. Archives of Institute of Civil Engineering, 2007.

[7] Barišić, I., Bogdan, M. & Dragčević V.: Application of Steel Culverts in Section of Osijek-Đakovo Motorway, Road and Rail Infrastructure, Proceedings of the Conference CETRA 2010 / editor: Lakušić, S. Zagreb, 17–18 May 2010, Opatija, Croatia, pp. 381-387

11 URBAN TRANSPORT

TEACHING ETHICS TO TRANSPORT ENGINEERS – THE RATIONALE BEHIND AND PRACTICE AT VIENNA UNIVERSITY OF TECHNOLOGY

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Abstract

Transport engineers' methods and designs alter existing structures. People's technological products create people as products of technology. From small-scale application of fair share policies to large-scale ubiquitous access policies, the ethics of transport and settlement systems play an important role. Too often, ethics are not considered appropriately, are overlooked or even ignored. However, engineers do not act detached from ethical values. A frequent practice among engineers is to adopt ethics of good intention instead of ethics of responsibility for their interventions. Engineers use models which represent only a window of reality. As engineers perceive what they have learned to perceive, education is of critical relevance. In our contribution we first carve out the dimension and importance of transport engineering ethics by referring to philosophers of technology including Jacques Neirynck, Günther Anders and Daniela Bailer-Jones and by giving examples. Adding to this we introduce the approach at the Research Center of Transport Engineering (IVV) at Vienna University of Technology (VUT) to engineering students' ethics education by providing three lecture series covering a very broad view on transport. Zooming in from general ethics concepts to mode-related points of view, these lecture series are: "Ethics and technology", "Barrier-free transport planning for public spaces" and "Cycling in the city". These lectures do not only provide an extended viewpoint from other engineering branches but also – which is most valuable – add perspectives from sociology, public health, law, philosophy and religion to transport planning.

Keywords: education, engineers, ethics, lectures

1 Introduction

Planners and engineers intervene in existing structures and shape them from nanotechnologies via light bulbs to nuclear power. Transport engineers are also part of the wide spectrum of engineers shaping the world. Philosophers of technology like Günther Anders [1] coined the notion that technology is not a neutral means to an end – free of virtue. But technology and its devices inherit determined usage by design. Specific economic, social and political conditions produce technology and devices that in turn entail economic, social and political changes. Human beings become as much a product of technology as technology is a product of humans. This notion is illustrated well in the energy flow density of the human being embedded in its socio-technological environment (see Figure 1).

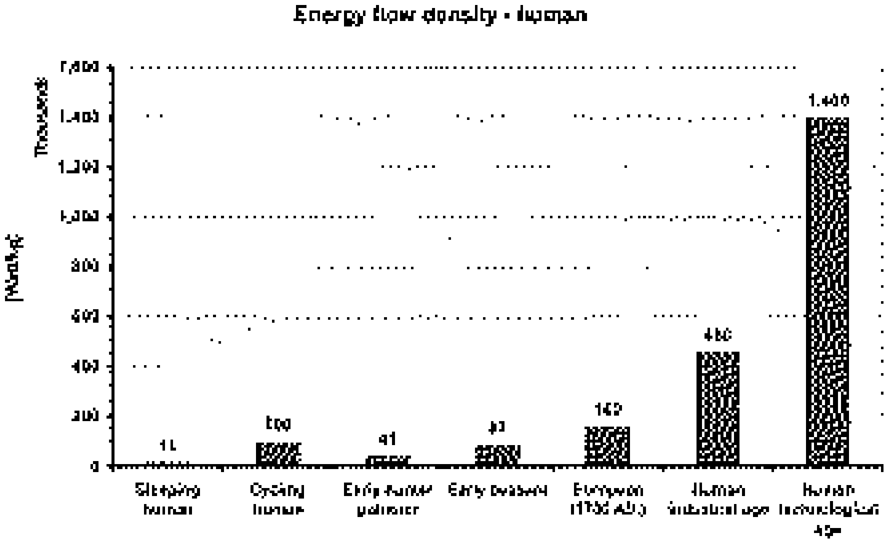

Figure 1 The energy flow density of the human in its socio-technological embedding. The more technology human beings utilize in their surroundings the more energy intensive become their lives [2]

2 Ethics and planning

The development of the classical natural sciences accompanied by technical disciplines was obviously much faster, than a responsible use of those new achievements was possible. The technical environment has (technically) expanded our physical abilities and sensory organs and thus exceeded our evolutionary experience areas [3]: e.g. during millions of years evolution designed our brain for walking speeds, but the couldn't possibly have adapted yet to the ever increasing speeds of transport means within the last 200 years.
The reaction to these changes and the social, economic, environmental and political consequences were not considered adequately. The consequences often even appeared surprising to many people. Another philosopher of technology, Jacques Neirynck, states that engineers show an infant understanding of ethics, not feeling responsible for their actions [4]. This notion is specified by Knoflacher [3] as ethics of good intention vs. ethics of responsibility. While the infant ethics of good intention are based on "meaning one's actions well", the adult ethics of responsibility are founded on "being accountable for one's actions". But today in transport planning ethics of good intention are still applied instead of ethics of responsibility, which would mean that transport engineers would apply the best of knowledge and would stand up for their designs and proposed solutions. Pure technical feasibility too often prevails over ethical evaluation of impacts. In transportation for example environmentally harming transport behaviors of the future are predetermine and prolongated by today's decisions for non-sustainable infrastructure like new roads or motorways. Unfortunately though, too often engineers retreat to modeling results and exterior causes, when transport engineering actions don't provide forecasted results. Due to increased computing power, engineers have put model results on a very high pedestal of relevance. But one has to be aware that models only represent assumptions central to model builders and users, which by definition need to be considered as true to work within the model's framework [5].
Not only do transport engineers build models of the world, they also massively shape the urban and natural landscapes (see Figure 2) and the everyday mobility behaviour of people.

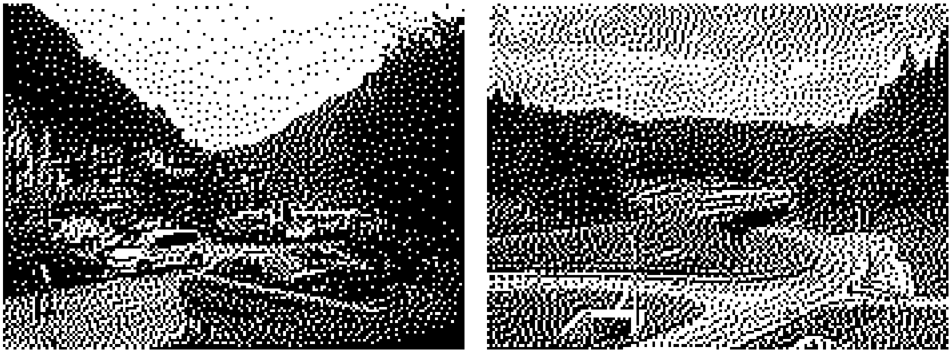

Figure 2 Transport engineers' products are massively shaping the urban and natural landscapes. Left: Parking lot dominating the central square of the UNESCO world heritage village Hallstatt (AUT). Right: national road, motorway and motorway junction ramps spare little of the quaint valley near Trojane (SLO). Photos: Brezina

So, figuratively speaking for engineers, finding the key (sustainable solutions to problems) is easier when the light beam (of knowledge and perspective) shines wider for them (see Figure 3). In this allegory ethical education provides a tool for increasing the illuminated area. With this perspective in mind it is therefore necessary to educate transport engineering students in ethical matters.

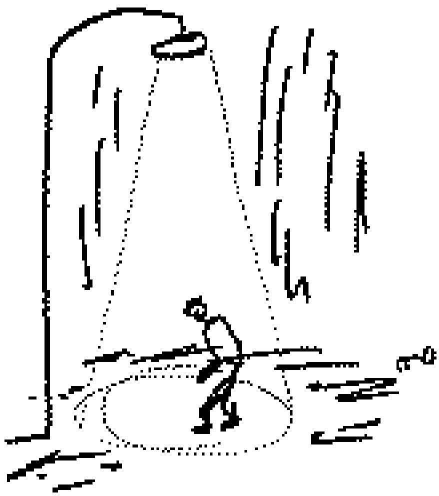

Figure 3 Streetlight effect: How to find the key (righ edge) outside of the light beam? Ethical education will increase the illuminated area. Drawing: Knoflacher

When asking what kinds of dimensions ethics education may have, IVV's educational concept offers a 3-step approach of ethical experience. The increasing likely power of effect on a planner increases from on to three:

1 The point of view of others. This includes other people and fields, often with a very different horizon and experience.
2 The point of view of the most fragile regular participant. The biggest Issue here is, how to incorporate this view systematically into planning.
3 Self-experience. Students will experience hands-on differences in design, e.g. with or without barrier-free planning.

3 Practice at VUT

Educating transport engineers from an ethical perspective has a long lasting tradition at the Research Center of Transport Planning and Traffic Engineering (IVV) of the Vienna University of Technology (VUT).

3.1 Base course in transport planning

In an early stage all civil engineering students at VUT need to attend the basic course (lecture and field work) in transport planning. Obligatory part of the field work is the one hour movement through the city with a baby stroller or a wheelchair. There students are at the mercy of good or bad design provided by active engineers (see Figure 4). This is the first hand opportunity for future engineers to experience, how small decisions about the design of the public space and public transport effect everyday lives of people with special needs.

Figure 4 Students of the base course on transport planning at VUT experiencing first hand the shortcomings and barriers in the transportation system from the point of view of the weakest participants. Photos: Hölesic

3.2 Lecture series "Ethics and technology"

It is astonishing that questions of ethics are not part of basic education at universities of technology. Questions of responsibility and reflective action could not be limited to technical disciplines, particularly because problems occur between the different disciplines. Interdisciplinary thinking and collective working is in demand for engineers to reduce differences and find better solutions. Therefore, we at IVV provide the lecture "Ethics and technology" since 2008 for students of all disciplines at VUT. The topics of the lectures are from various disciplines reaching from natural sciences, technology, social sciences up to religious studies and politics. More than 150 students attend the lectures every year since. The ever-increasing number of students confirmed the importance of this topic for the future graduates of technological studies.
The lectures address the issues of, for instance, the interaction between technology and ecology. This is exemplified by the topic of climate change, technology assessment and ecological principles – shown by the assessment of consequences of the application of engineering and technology in nuclear energy. The lectures examine the question of how engineers and technicians can act in everyday life ethically at all. In this case, the categorical imperative of Immanuel Kant is in the focus of discussions: "Act only according to that maxim whereby you can at the same time will that it should become a universal law without contradiction."[6] This rule can be also found in modified form in the principles of the five world religions.

The lectures give an overview and show that common fundamental ethical principles exist. At the end of each lecture, the students have to take part in a discussion about the content of the presentation. Thereby they learn to deliberate, to argue and to develop their own opinion. The discussions show that it is becoming more and more difficult to accomplish a comprehensive principle of responsibility (in dubio pro malo)[7] according with the global economic dependencies and an increasing constraint of technical producibility.

3.3 Lecture series “Barrier-free design of public space”

Recently carried out research studies (EGALITE [8] and EGALITE-Plus [9]) have tried to identify groups of mobility impaired people and to quantity the development of these mobility impaired people in Austria till the year 2050. The findings of these studies were alarming. One result for example was that more than 40% of the total population of Austria is temporally or permanently handicapped and this share is even expected to increase till 2050. At the same time when these studies were carried out, the IVV was procured to carry out a study called GABAMO [10] where the education sector in Austria regarding planning for mobility impaired people was under scrutiny. In total 93 bachelor and master courses with 654 lectures where investigated and analysed regarding their contribution towards barrier-free planning. It turned out that at Austria’s universities and also at the so called “Fachhochschulen” no dedicated master course for barrier-free planning exists and even worse, no dedicated lectures for barrier-free planning are offered nationwide.
Based on the findings of the afore mentioned studies, IVV made the decision to offer an explicit lecture for barrier-free planning of public space open for students of the civil engineering, architecture and land-use planning faculties. The first lecture took place in the winter semester 2012/2013 and since then, it has been repeated every year. In the starting year about 20 students took the opportunity to learn about planning principles for barrier-free design [11]. The lecture’s contents are:

- existing national and international norms and guidelines dealing with barrier-free design of public place;
- discussion with international well known experts on barrier-free public transport station building designs;
- the application of the MofA-method (developed in Vienna) [12] to assess the barrier-freeness of public space and public transport station buildings.

The lecture seems to have become a success story because in the second year we had to limit the number of students to 50 persons due to capacity restraint reasons.
Summarising it can be said that universal design and design for mobility impaired people groups is becoming more and more important, not only in Austria but also worldwide. An aging population combined with climate change and scarceness of non-renewable resources will lead to a significant change of our future ways of living and mobility patterns. And our future planers will need to have the skills to design the built environment so that all members of the society can fulfill their mobility needs. With our lecture “barrier-free planning for the public space” we help these planners to gain these necessary skills.

3.4 Lecture series “Cycling in the city”

The third and youngest lecture with a strong ethics background at IVV is the lecture series “Cycling in the city”, where on 13 to 15 evenings experts from the most diverse disciplines concerned with cycling provide the “other field’s point of view”. This lecture is open to students of all fields, but a numerous general audience attends too. Established in the year 2013, which was proclaimed as the year of cycling by the City of Vienna, the lecture goes in the second season this year. And we managed to win over new and international lecturers, providing

engineering students with new perspectives from disciplines revolving around cycling. Urban planners, transport engineers, historians, sociologists, physicians, psychologists, lawyers, designers, philosophers, mechanical engineers, marketing and advocacy professionals and public administrators stimulate the discovery of new facets of urban cycling.
The first season proved to be more popular among students and general audience than ever expected. On average 255 students (min: 185, max: 287) attended the lectures (see Figure 5), so that the talks had to be live streamed to a second lecture room to meet the demand.

Figure 5 Overcrowded lecture room at the "Cycling in the city" lecture". Photos: Brezina/Schumich

4 Conclusion

Even though there's hardly any tradition in ethics education to be found at universities of technology, the success of IVV's lectures shows that students are highly interested in adding ethical perspectives to their mental toolboxes where otherwise purely engineering tools would dominate. We therefore strongly recommend that ethics related courses are provided at departments with technology focused studies at other universities – especially where transport engineers are educated.

References

[1] Dries, C.: Technik als Subjekt der Geschichte? Technik- und Gesellschaftsphilosophie bei Günther Anders. Darstellung und Kritik., in: Philosophische Fakultät, Albert-Ludwigs-Universität, pp. 132, 2004.

[2] Chaisson, M.: Cosmic evolution – The rise of complexity in nature, Harvard University Press, 2001.

[3] Knoflacher, H.: Evolution – Ethik – Technik (Chapter), Ethik und Technik, H. Knoflacher, et al., Editors, Lit-Verlag: Wien, pp. 35-55, 2009.

[4] Neirynck, J.: Der göttliche Ingenieur. Die Evolution der Technik, Expert-Verlag GmbH, 2001.

[5] Bailer-Jones, D.M.: Scientists' Thoughts on Scientific Models, Perspectives on Science, 10 10(3), pp. 275-301, 2002.

[6] Kant, I.: Grounding for the Metaphysics of Morals – with: On a Supposed Right to Lie because of Philantropic Concerns, 3rd (1993) ed, Hackett Publishing Company, Inc., 1785.

[7] Schulze, S.: Die Etablierung der Bioethik, Philopedia. http://www.uni-potsdam.de/ethik-online/index.php/24-teilbereiche/bereichsethiken/bioethik. Universität Potsdam, 10.02.2014.

[8] Graf, W., Meschik, M., Meth, D., Neumann, A., Sammer, G., Unbehaun, W., Millonig, A., Schechtner, A., Aleksa, M., Maurer, P., Saleh, P., Stütz, R., Dangschat, J.S. & Pfaffenbichler, P.C.: ÉGALITÉ – Ein gleichberechtigter Alltag im telematikgestützten Verkehrsgeschehen. 2006

[9] Sammer, G., Uhlmann, T., Unbehaun, W., Millonig, A., Mandl, B., Dangschat, J. & Mayr, R.: Identification of Mobility-Impaired Persons and Analysis of Their Travel Behavior and Needs, Transportation Research Record: Journal of the Transportation Research Board, 2320 2320, pp. 46-54, 2012.

[10] Schopf, J.M., Haller, R., Lemmerer, H. & Mayerthaler, A.: Aus- und Weiterbildung im Bereich der barrierefreien Mobilität. Beiträge zu einer ökologisch und sozial verträglichen Verkehrsplanung, ed. H. Zukal, Vol. 1/2011, Institut für Verkehrswissenschaften – Forschungsbereich für Verkehrsplanung und Verkehrstechnik, 2011.

[11] Emberger, G., König, I., Krpata, R., Rollinger, W. & Grundner, M.: Public transport for (disabled) people – the Vienna experience, 13th World Conference on Transport Research (WCTR), Rio de Janeiro, pp. 20 (Paper nr. 2913), 2013.

[12] FFG: MofA – Mobilität für alle – mobility for all. http://www2.ffg.at/verkehr/projekte.php?id=595&lang=en&browse=programm. FFG

INNOVATIVE APPROACHES OF PROMOTING NON-MOTORIZED TRANSPORT IN CITIES

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Abstract

As the number of cyclists on Viennese streets grows, they are increasingly seen as a danger to pedestrians and even car drivers, especially by the media. Statements like "these scofflaws don't obey the rules" or "they run red lights and don't stop at stop signs" are common. Public perception is that also pedestrians tend to re-interpret traffic laws in their sense – they simply cross intersections against red lights when they feel safe about it (physically, not only in terms of getting caught). The most common reaction to this behaviour is the claim for more enforcement and even license plates for bikes. We propose a more innovative approach, which takes into account the reasons for this legally non-conformant behaviour of non-motorized traffic: bikers and pedestrians should be allowed to run red lights when not obstructing or endangering themselves or others. Under the same circumstances, cyclists should be allowed to treat stop signs as yield signs. But why?
"The purpose of all the traffic lights, signs, and lines – is to prevent cars from running into everything else" [1]. Current laws are ensuring the ease and flow of motorized traffic often at the expense of the ease, flow and even safety of non-motorized traffic. Motorists must obey these laws due to the fact that they are impaired in their visual and acoustic perceptions by the drivers' perspective. Cyclists and pedestrians on the other hand can run red lights and stop signs without safety concerns – they have a better, unobstructed view on cross sections, they can accelerate and brake within fractions of a second, they can hear even quiet safety hazards, have little inertia and a low potential for damage. In this paper we analyse which built and legal structures would be necessary to make a city work without car-oriented regulations and what it would look like.

Keywords: red lights, jaywalking, scofflaw, self responsibility

1 Introduction

As the number of cyclists on Viennese streets grows, they are increasingly seen as a danger to pedestrians and even car drivers, especially by the media. Statements like "these scofflaws don't obey the rules" or "they run red lights and don't stop at stop signs" are common. Public perception is also that pedestrians tend to interpret traffic laws rather broadly – they simply cross intersections against red lights when they feel safe about it (physically, not only in terms of getting caught). The most common reaction to this behaviour is the claim for more surveillance [2] and even license plates for bikes [3].
We propose a more innovative approach, which takes into account the reasons for this legally non-conformant behaviour of non-motorized road users: bikers and pedestrians should be allowed to run red lights when not obstructing or endangering themselves or others. Under the same circumstances, cyclists should be allowed to treat stop signs as yield signs.

In the second section we analyze Viennese traffic accident statistics, focusing on accidents at signalled and non-signalled intersections. In section 3 we look at the question why regulations are disregarded in the first place, and why there is a difference between car drivers and active modes (pedestrians and cyclists). In section 4 we propose new Road Traffic Regulations which better respect the needs and abilities of different transport modes. Section 5 gives international examples where parts of our claims are already successfully implemented. In section 6 we sum up our experiences in proposing these new regulations and draw conclusions in section 7.

2 Analysis of red walkers and accidents at Viennese intersections

The Viennese modal split is currently (2011) 28.3% walking, 5.6% cycling, 37.2% public transport and 28.9% motorized individual transport, with the goal of 27% (sic!) walking, 8% cycling, 40% PT and 25% motorized individual transport until 2020 [4]. The Viennese cycling network is about 1,250 kilometres, consisting mostly of bike routes (55%, only indicated with signs), marked cycle lanes (24%, painted on the road) and separated cycle paths (21%) [5].

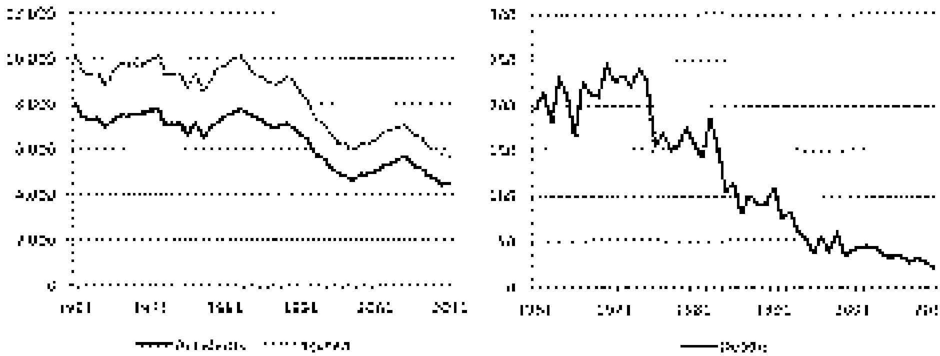

Figure 1 Time line of number of traffic accidents, injured (left) and deaths (right) in Vienna [6]

The number of traffic deaths in Vienna has drastically decreased in the last 50 years and only stagnated in the last decade at about 20 to 30 per year. The number of accidents and the number of injured are closely correlated and have not decreased that much (see Figure 1). When looking at the distribution of means of transport for casualties, severely injured and deaths, it becomes clear that pedestrians and cyclists as most vulnerable road users contribute to traffic deaths above average (see Figure 2). For 2020 the Viennese government has issued the goal of Vision Zero – zero fatalities in traffic in 2020 [7].
As pedestrians are overrepresented in the number of deaths compared to their modal split and also their number of accidents, special efforts are undertaken to increase pedestrians' safety. Red walking is regarded as risk factor and has been surveyed by the Austrian road safety association Kuratorium für Verkehrssicherheit (KfV) in 2002 [8].
9 Viennese intersections were observed and red walkers were questioned about the motivation for their actions. The study found that at these intersections on average 81.1% obeyed the traffic lights and walked on green. Of those crossing the intersections on red, about 18% were red runners, trying to catch a tramway or a bus, thus probably not exerting special attention to traffic. 82% however walked against the red light deliberately. Of all red crossers over 50% crossed shortly after the light changed to red (late starters) or shortly before the light changed to green (early starters) thus using the safety reserves of the traffic light signalization.
The KfV authors have tried to get further insight into the red walking topic by analyzing the Viennese accident data 1992 to 1998. They found that around 80% of pedestrian accidents at signalled intersections were caused by cars, another 9% by lorries. Unsurprisingly only

3% of pedestrians were not injured while more than a quarter were severely injured and over 2% died. The analysis of accident causes showed that vehicle drivers were to blame in more than half of the pedestrian accidents at signalled intersections with crosswalks. And vehicle drivers are also accountable for nearly all pedestrian accidents at unsignalled crosswalks as pedestrians have the right of way there.
To conclude, even when pedestrians obey red lights and keep to marked crosswalks they cannot rely on not being harmed. The traffic light drill exerts a sense of “pseudo-safety”.

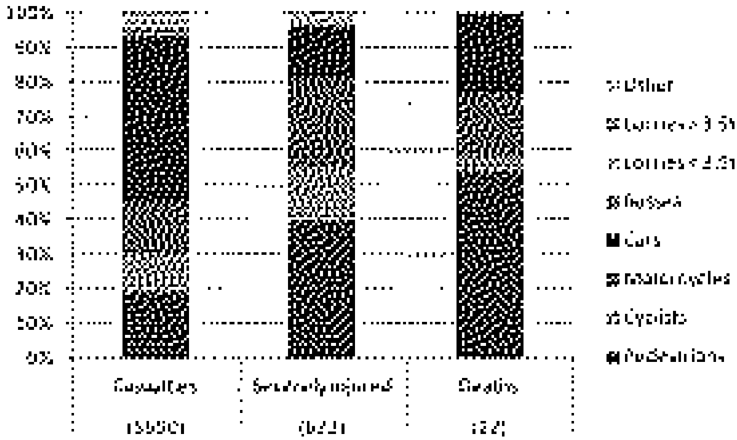

Figure 2 Distribution of means of transport for casualties, severely injured and deaths from traffic accidents in Vienna 2011 (total number in brackets) [6]

3 Understanding rule-breaking behaviour and differences between means of transport

3.1 Why do pedestrians and cyclists often disobey the rules that should protect them?

“The purpose of all the traffic lights, signs, and lines – is to prevent cars from running into everything else” [1]. Road Traffic Regulations were not designed for pedestrians and cyclists in the first place. They were introduced (by powerful lobbies – see criminalization of “jaywalking” [9]) to ensure the ease and flow of motorized traffic, often at the expense of the ease, flow and even safety (see section 2) of non-motorized traffic.
Furthermore, traffic lights are commonly oriented on the traffic volumes of motorized traffic, distributing green times according to the number of cars. Pedestrians are often left with remaining time gaps and long waiting times. Separated cycling paths and pedestrians are often signalled by combined traffic lights oriented on the clearance time of slow pedestrians. Obviously cyclists are discriminated by this arrangement.
So pedestrians and cyclists are discriminated by traffic lights that insufficiently protect them. The obedience of red lights is further diminished when traffic volumes are low and pedestrians would easily find a time gap to cross the street (correlation of -0.73 between traffic volumes and share of red walkers – without early and late starters) [8].
Finally, non-motorized “rule-breakers” almost never receive negative feedback; they nearly never get caught and penalized. On the contrary, walking against red lights saves time and may be safer than inattentively crossing a road on green. The choice of running red lights seems to be a cost-benefit-analysis with the benefit of saving time and exercising the right of self-determination (freedom), while the costs are possible safety threats and the risk of receiving a fine.

3.2 Why can pedestrians and cyclists (safely) disobey rules while motorists can't?

The cost-benefit-consideration provides an explanation to the disobedience of Road Traffic Regulations by pedestrians and cyclists, but it does not explain why motorists practically do not run red lights. The license plate as a means of identifying rule-breakers is often regarded as a measure to increase compliance. License plates for bicycles are by popular opinion believed to ensure the obedience of red traffic lights [3]. But the fact that car drivers violate many other e.g. parking regulations despite having a license plate leads us to question whether traceability is the sole barrier in running red lights.
Obviously there must be some transport mode inherent characteristic that restrains car drivers from crossing against red lights but does not hinder pedestrians and cyclists from doing so. We believe that the ability for perception of the surroundings is crucial.
Pedestrians and cyclists have an unobstructed perception of the environment; they see, hear and even smell their surroundings directly. By having little mass and inertia they are very manoeuvrable. They can quickly accelerate and decelerate. Their line of sight is elevated in contrast to a seated car driver's.
Car drivers on the other hand only have an obstructed view. Through their drivers' perspective they have a low seating position, a limited field of vision and blind spots. Their acoustic perception is obstructed through closed windows or even loud music. Enormous mass and inertia make them rather inert (in terms of reaction) and inflexible road users.
And finally, pedestrians and cyclists are vulnerable. As unprotected, "weak" road users they are at the mercy of motorists, even when they have the right of way. And if they decide to disobey a red traffic light, they mostly exert special care and attention (92% look left and right at a red signal before crossing vs. 67% at a green signal [10]) as they will be most probably the only casualties in an eventual accident.

4 New Road Traffic Regulations

Now that we have identified the existing regulations as discriminating for non-motorized road users and argued that disobeying some of these rules does not necessarily result in an increased number or severity of accidents, we propose a new set of traffic regulations. Following the Viennese goals of Vision zero and target modal split, we deduce measures how to reach those goals. A core principle of the new Road Traffic Regulations should be the promotion of self responsibility and consideration (protection of others). We propose to legalize crossing against red lights for pedestrians and cyclists when not obstructing or endangering themselves or others. Under the same circumstances cyclists should be allowed to treat stop signs as yield signs. To improve the conditions for pedestrians and cyclists, traffic lights should be reduced drastically and replaced by pedestrian and cyclist crossings. As this measure alone does not ensure the safety of non-motorized traffic, the general speed level within the city should be reduced and intersections should be decelerated by design e.g. by raising the intersection plateau or narrowing the cross section.
At the remaining signalled intersections the green times should be designed for active mode needs or at least the turnaround times should be minimized to reduce pedestrians' waiting times.

5 International examples

Depending on the exact wording in the road traffic regulations, walking on red lights is at least not forbidden in some Europeans countries. E.g. in GB, §21 of the Highway Code advises that one "should only start to cross the road when the green figure shows", in contrast to §34 where one "MUST NOT cross or pass a stop line when the red lights show" (emphasis in original) at railway level crossings [11]. According to the Norwegian traffic rules, a red signal means that

one should not start crossing the road when it is not possible without obstructing traffic or involves danger [12]. As no accumulation of accidents is known from these countries, future implementation in Austria could be regarded as safe as in these countries.
For cyclists, right on red lights (at marked intersections) is an uprising concept partially already implemented (as in Hamburg/Germany [13]) and partially in the evaluation phase (as in Brussels/Belgium [14], Paris/France [15] and Basel/Switzerland [16]). No negative effects are known from this measure that allows cyclists to turn right at dedicated intersections even when facing a red light. In the State of Idaho, the Idaho Stop Law is in effect since 1982, allowing cyclists to treat stop signs as yield signs and red traffic lights as stop signs [17]. Also no increase of accidents could be seen there. On the contrary, research shows that cycling is safer in places where the Idaho Stop Law is in effect [18].
Right turn on red (RTOR) is legal for cars in many states of the US since the 1970s [19]. It was implemented during the Oil Crises to save fuel and travel time [20] despite the fact that it increased accidents with pedestrians and cyclists by up to 100% [21]. This indicates that the concept is not as feasible for cars as it is for bikes and that in the USA economic considerations dominate over safety concerns.

6 Discussion in Austria

Ulrich Leth first proposed the idea of legalising red walking in a carefully argued article in an Austrian nationwide newspaper in November 2013 [22]. One week later, public service television and radio took on the topic often reducing the message to “pedestrians should generally be allowed to cross against red lights”, which led to a public outcry. Nearly all political parties, both automobile lobbying organisations, the Ministry of Transport and the Austrian road safety association opposed the idea without even seriously considering it [23][24][25]. Not to mention various personal verbal attacks against the author.
The three major reasons for opposing the proposal were: (1) Uncertainty about the accountability of the car driver in case of an accident. This could be easily resolved by reformulating the road traffic regulations accordingly (as in e.g. Great Britain). (2) Working international examples cannot be transferred to Austrian conditions, critics say. (3) The role-model function for children is undermined when adults walk on red lights. This appears to be the only reasonable argument. However, already now children are excluded from the “principle of legitimate expectations” valid in Austria, meaning that a car driver has to exert special care any time approaching a child in traffic. Society should aim at providing a child-friendly traffic rather than produce traffic-friendly children [26]. At least the cycling commissioner for Vienna and the cycling lobby supported the idea of RTOR for cyclists [27][28]. We have learned from this discussion that despite the weaknesses of the current traffic rules (disobedience, lack of safety, etc.) are known, innovative ideas are not welcome and will be instinctively opposed in the first reaction.

7 Conclusion

We started by analyzing a study about red walkers’ motives and pedestrian accidents in Vienna concluding that neither green traffic lights nor marked crossings provide pedestrians and cyclists with the intended safety. Partly because of this, partly due to the fact that pedestrians are discriminated by short green times and long waiting times, red lights are often ignored. We propose to legalize this behaviour and reimplement self-responsibility in the road traffic regulations. Pedestrians and cyclists are able to cross red lights and stop signs without safety concerns as they have a direct, unobstructed perception of their surroundings. International examples show that this is feasible and safe. Reactions in Austria show that it is still a long way to go.

References

[1] TheWashCycle – The Myth of the Scofflaw Cyclist, http://washcycle.typepad.com/home/2008/07/the-myth-of-the.html, 10.02.2014.

[2] The Ney York Times – In a Car-Culture Clash, It's the Los Angeles Police vs. Pedestrians, http://www.nytimes.com/2013/12/26/us/in-a-car-culture-clash-its-los-angeles-police-vs-pedestrians.html, 10.02.2014.

[3] Schmitten, P.: Zur Diskussion: Kennzeichenpflicht für Fahrräder?, in mobilogisch!, Zeitschrift für Ökologie, Politik und Bewegung, Heft 2/2013, http://www.mobilogisch.de/aktuell/41-ml/artikel/153-kennzeichenpflicht-fuer-fahrraeder.html, 23.02.2014.

[4] Masterplan Verkehr Wien 2003 – Evaluierung 2013. Werkstattberichte 135, Magistrat Wien, MA 18, Stadtentwicklung und Stadtplanung, 2013, http://www.wien.gv.at/stadtentwicklung/studien/pdf/b008353.pdf

[5] Zahlen und Fakten zum Wiener Radverkehrsnetz, http://www.wien.gv.at/verkehr/radfahren/radnetz/fakten.html, 10.02.2014.

[6] Verkehrsunfallstatistik 2011, Kuratorium für Verkehrssicherheit, http://www.kfv.at/unfallstatistik/index.php?id=57, 23.02.2014.

[7] Ziele – Verkehrssicherheitsprogramm Wien 2005 bis 2020, http://www.wien.gv.at/verkehr/verkehrssicherheit/programm/ziele.html, 10.02.2014.

[8] Robatsch, K., Mailer, M. & Franzl, E.: Rotgeher. Pilotstudie zur Problematik der Fußgängerquerung bei rotem Licht auf lichtsignalgeregelten Kreuzungen, 1. Auflage, Kuratorium für Verkehrssicherheit, 2002.

[9] Goodyear, S.: The Invention of Jaywalking, in: The Atlantic Cities, 24.04.2012, http://www.theatlanticcities.com/commute/2012/04/invention-jaywalking/1837/.

[10] Van Waeg, G.: Training our children – to wait for green, or – to cross when safe?, http://www.call.walk21munich.com/presentations/BS202_GeertvanWaeg.pdf, 23.02.2014.

[11] Rules for Pedestrians (1 to 35). The Highway Code, https://www.gov.uk/rules-pedestrians-1-to-35, 22.02.2014.

[12] Forskrift om offentlige trafikkskilt, vegoppmerking, trafikklyssignaler og anvisninger (skiltforskriften), http://lovdata.no/dokument/SF/forskrift/2005-10-07-1219/KAPITTEL_12#%C2%A724 (Norwegian), 22.02.2014.

[13] Informationen zum grünen Pfeil, http://www.hamburg.de/start-gruener-pfeil/, 22.02.2014.

[14] La région bruxelloise teste le tourne-à-droite cycliste, http://www.gracq.be/NEWS-2012-06/InaugurationTourneADroite, 23.02.2014.

[15] Au feu rouge, je passe?, http://www.paris.fr/pratique/deplacements-voirie/velo/le-tourne-a-droite/rub_9931_stand_114216_port_24784, 22.02.2014.

[16] Pilotversuch für velofreundliche Lichtsignalanlagen gestartet – Basel nimmt schweizweit Vorreiterrolle ein, http://www.medienmitteilungen.bs.ch/showmm.htm?url=2013-06-17-bd-001, 22.02.2014.

[17] Idaho Statutes, Title 49 – Motor vehicles, Chapter 7 – Pedestrian and bicycles, 49-720, http://legislature.idaho.gov/idstat/Title49/T49CH7SECT49-720.htm, 23.02.2014.

[18] Meggs, J.: stops as yields – safety and effects of the Idaho law, http://www.ecf.com/wp-content/uploads/Meggs-J-Stops-as-yields.pdf, 23.02.2014.

[19] 42 U.S. Code § 6322 – State energy conservation plans, http://www.law.cornell.edu/uscode/text/42/6322, 22.02.2014.

[20] Right turn on Red!, http://energy.gov/articles/right-turn-red, 22.02.2014.

[21] The Effect of Right-Turn-On-Red on Pedestrian and Bicyclist Accidents. Final Report. US Department of Transportation, Oct. 1981, http://ntl.bts.gov/lib/25000/25600/25603/DOT-HS-806-182.pdf, 23.02.2014.

[22] Rote Ampeln sollten nur noch für die Autos gelten, in Die Presse, 14.11.2013, http://diepresse.com/home/1479801/Rote-Ampeln-sollten-nur-noch-fur-die-Autos-gelten.

[23] Bei Rot über Kreuzung sorgt für Kritik, in ORF.at, 22.11.2013, http://wien.orf.at/news/stories/2616181/.

[24] „Bei Rot über Kreuzung wäre sehr gefährlich“, in Kurier, 22.11.2013, http://kurier.at/chronik/wien/bei-rot-ueber-kreuzung-waere-sehr-gefaehrlich/37.076.717.

[25] Rot oder rascher tot? Das Rotlicht soll Rotlicht bleiben!, in Die Presse, 10.12.2013, http://diepresse.com/home/meinung/gastkommentar/1502355/Rot-oder-rascher-tot-Das-Rotlicht-soll-Rotlicht-bleiben.

[26] Rote Ampeln nur für Autos, in mobilogisch!, Zeitschrift für Ökologie, Politik und Bewegung, Heft 1/2014.

[27] Radfahrer: „Abbiegen bei Rot in einem Pilotprojekt testen“, in Der Standard, 13.12.2013, http://derstandard.at/1385171021588/Radfahrer-Abbiegen-bei-Rot-in-einem-Pilotprojekt-testen.

[28] Ja zu Rechts bei Rot nach internationalem Vorbild, Radlobby, 28.11.2013, http://www.radlobby.at/cms/artikel/datum/2013/11/28/ja_zu_rechts_bei_rot_fuers_radl_nach_internationalem_vorbild.html.

PUBLIC PARTICIPATION FOR SUCCESSFUL TRAFFIC AND TRANSPORT PLANNING

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Abstract

In planning and building transportation infrastructure, citizens are increasingly demanding more information and rights of participation. This contribution for CETRA 2014 illustrates the state of the art of public participation in Germany and shows why participation makes sense, who should take part in the process of planning, at which point, and the methods that should be used. The aim of public participation is to improve the involvement of citizens in planning. It is important to increase the appreciation of citizenship, politicians and administration for each others' concerns and duties. For traffic and transport planners, it is useful to get ideas and proposals from experts of 'everyday life'. Experience shows, that public participation positively influences the planning process and improves the level of acceptance for planning projects. For the procedure of participation, the following three target groups have been identified, each with a different role and professional competence: institutional stakeholders (administration, parliamentary groups etc.), citizen and lobby groups (citizens´ action groups, lobbies etc.) and the general public (people living in the planning area and other concerned persons). The different forms of participation have to address the interests and capabilities of the different groups. The simplest way of participation is the transmission of information to the citizens and the exploration of their interests and opinions. In both cases communication only runs in one direction: from planner to citizen or vice versa. In more intensive forms of participation a two-way exchange occurs. The most intensive type of participation is one where co-operation takes place, where citizens and professionals work together. During the last few years different methods of public participation have been developed. In present-day Germany citizen forums and workshops are wide-spread. For concrete tasks of urban planning, 'planning factories' have been used. Participation via electronic communication media is quickly gaining in importance.

Keywords: publicity, traffic planners, citizen, policy, public participation, communication, mediation, media, e-participation

1 Introduction

Public attitude concerning the build of new road and railways has changed in the last decades. Many citizens judge large building projects critically and are protesting against them. The motivations for such protests are versatile:
- disapproval of the building of transportation infrastructure in one's neighbourhood;
- environmental and social concerns: protection of natural and cultural assets;
- discontent with traffic policies that support highspeed railways and highways;
- desire to use public funds for other purposes;
- suspicion of federal plans and projects;
- disapproval of an enforcement related policy – and planning style.

Citizen protests frequently result in project delays due to changes in procedures and prosecution on the one hand. On the other hand the protests divide the people and the politicians. In the time period between 2009 to 2011 the German project “Stuttgart 21” has attracted a high level of public attention. The existing above-ground, dead end main station in the middle of the major city Stuttgart (approx. 600.000 citizens) is supposed to be replaced with an underground railway station. Ever since the unveiling of the plans, a sizeable alliance consisting of citizens and NGOs have protested vehemently against the project [1]. As a result “Stuttgart 21” has became a highly sensitive issue in German domestic policy.
The protests against “Stuttgart 21” have, in affect, forced the academic and policymaking world to take a hard look at public participation, resulting in several publications with policy recommendations. Examples are the “guide for a proper public participation” from the Federal Ministry of Transportation [2] and the social science-based publication “in-time public participation for efficient traffic infrastructure planning” [3].
This paper illustrates the state of the art of public participation in Germany. The article explains why public participation is necessary and who should be engaged to apply the correct method in the appropriate time frame. This paper relies on an article entitled “advice for participation and cooperation in traffic planning” [4], published by the FGSV (German Road and Traffic Research Association), and written by this author, who is also a member of the relevant task force.

2 Goals of public participation

Major goals of public participation are in fact:
- Strengthening of participation and transparency: plans should not be made “on the sly” behind citizens backs.
- Encouragement and understanding between citizens, policy makers and administration of the relevant concerns or tasks: knowledge of the motives of other parties reinforces acceptance.
- Applying ideas and proposals based on the view of the user (knowledge acquired from daily life) makes fieldwork easier for the planner.
- Positive influence on planning process and preparation of social and political acceptance: experiences show that public participation leads to an improved understanding and a higher level of acceptance of the project.

To achieve these aims some requirements have to be considered. One important fact is that the “rules of the game” are clearly defined to the citizens. For instance, it must be clarified that the public only has advisory function. All decisions concerning the project are finally made by the democratic elected parliament. Continued participation at early stage is also very important: helpful suggestions of citizens can only be considered properly in the early course of project. If citizens are informed of plans after they have been finalized, resistance is practically inevitable . Furthermore, the different groups of society must be addressed and involved in the participation. Based on past experiences, women, children and adolescents, employees and migrants are mostly underrepresented. Therefore, these groups deserve special focus and participation procedures should be customized to meet their needs.
In conclusion, sufficient resources must be made available for the procedure of participation. Although participation may require financial resources, by taking the proper precautions, other financial expenses, such as legal expenses from lawsuits, can be avoided.

3 Stakeholders in public participation

There are three different groups of addressees who differ from each other in their role and expertise (Figure 1).

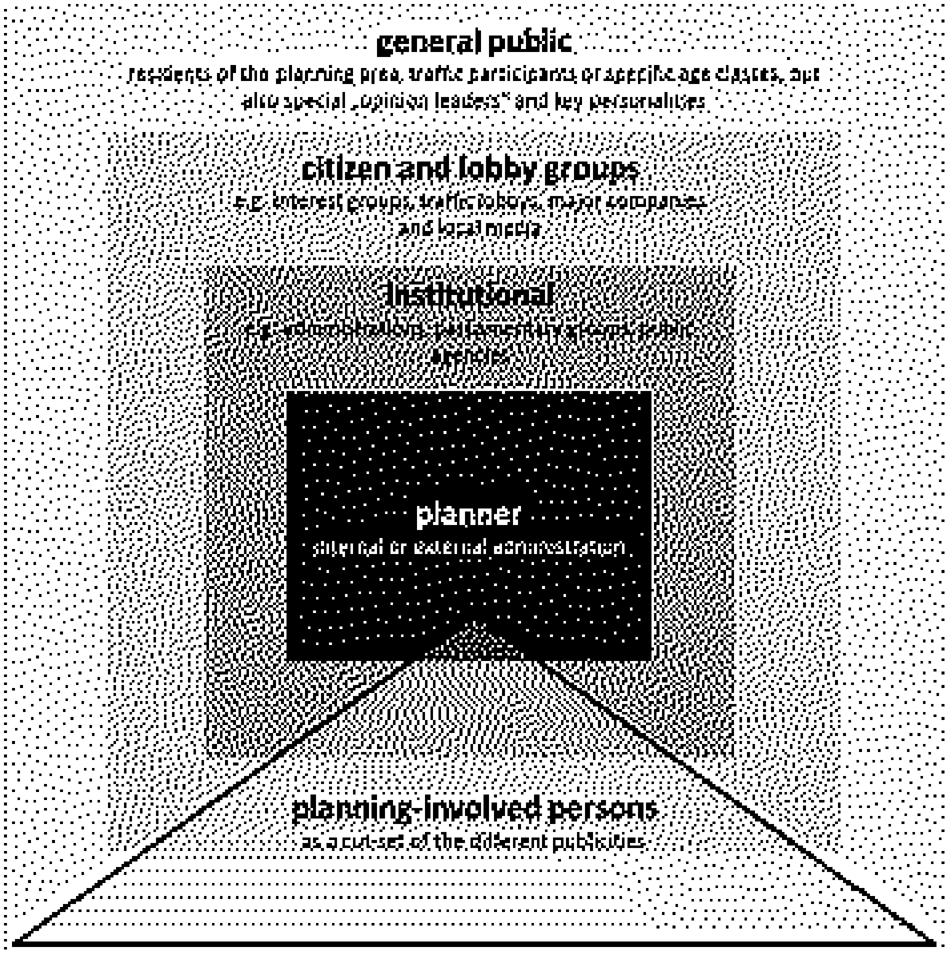

Figure 1 Public in traffic planning [4]

3.1 Institutional

Agencies, administrations and political committees belong to the institutional stakeholders. They are traditionally embedded in the planning process and deal with professional traffic planning tasks.

3.2 Citizen and lobby groups

Citizens and lobby groups can be summarized as organized groups. Mostly, they consist of voluntary representatives. Although they consist predominantly of laymen, many citizens' initiatives have developed excellent expertise.

3.3 General public

General public includes both, residents as well as further interested and concerned persons. They consist predominantly of volunteers. This has to be considered by using a common, coherent type of language and presentation.

4 Different types of public participation

The different types of participation have to cover the interests and options of the different groups. The basic purpose of participation is first, to elicit information and opinions from citizens, and second, to inform citizens about the current status of a planning project. In both instances the communication channel runs in one direction, from planners to citizens or vice versa. The purpose of gathering information and opinions is to understand attitudes, evaluate the level of knowledge and the behaviour of citizens. Thus, hints to conflicts and potential cooperation in planning can be extracted, especially at the beginning stages of planning.
Informing citizens is important for imparting the goals and contents of planning and gives citizens the opportunity to form their own opinions. Accordingly, it is fundamental to communicate which parts of the planning process are only loose ideas and which are already concrete determinations – in every stage of planning.
During public participation sessions, an exchange takes place between planners and citizens. By expressing their demands and opinions in hearings or assuming the role of a planner in task forces, they take an active part in the planning process.
The most intensive type of participation is a cooperation, where professionals and laymen work together to solve problems. Intensive communication, working towards a common denominator and compromises are the significant attributes of this type of participation.

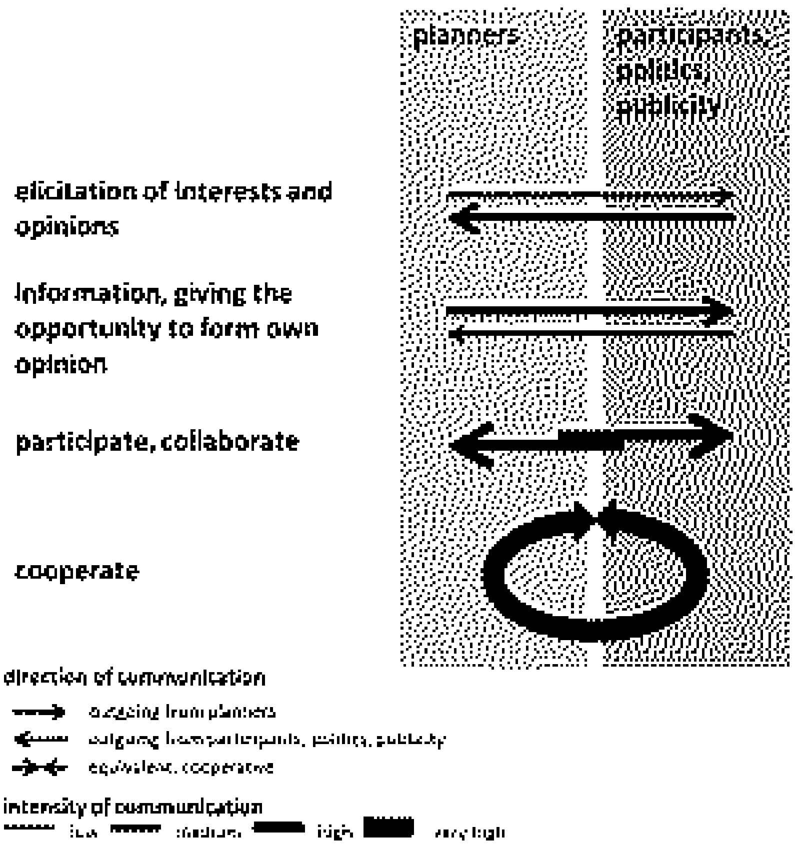

Figure 2 Different participation and cooperation types [4]

New types of participation and cooperation will evolve with the expansion of the internet and modern media. On the one hand these new technologies increase the possibilities for reaching the general public and eliciting feedback. On the other hand there are risks associated with the speed and ease at which information is transmitted. For instance, these new communication forms are so dynamic that it is becoming more difficult to prevent the spreading of false information.

5 Techniques of public participation

During the past few years, many different techniques for participation have developed, which are suitable for various types of participation at different planning stages. Citizen surveys are very useful for eliciting opinions, especially at the beginning stages of planning. Quantitative methods in form of questionnaires as well as qualitative surveys (interviews with individuals or groups) are useful in this regard. In several cities complaint management systems have been implemented, the results of which provide important feedback for future planning.
For information and opinion making, different types of media can be used: websites, newspapers, flyers for households, posters, exhibitions and so on. Open councils allow planners to go into greater detail with their plans and to address emerging questions from citizens. Excursions through the city can be very helpful as well, because plans can be explained better while on location. In contrast to using maps, excursions give citizens a real-life example of completed projects.
As far as types of participation, citizen forums and citizen workshops are the most prevalent in Germany. Together, citizens and experts discuss plans to develop their own solutions. A new possibility of participation is e-participation. In chats and newsgroups citizen are able to evaluate solutions, to communicate their ideas and comment on suggestions of others. One example of this is the online participation in the transport development plan of Bremen [5].
Further types of participation are roundtables as well as mediations. Their main goal is finding solutions for existing and foreseeable conflicts in planning.

6 Conclusion

Public participation is an important part of traffic planning processes and is gaining increasing influence. Awareness about the importance of public participation is prevalent in Germany. In a current survey, for example, the Federal Association of German Industry and other marketing associations advance the view of a proper and comprehensible information of the citizen about projects. They also recommend an intensive participation at earlier stages of planning [6]. In recent strategic traffic development planning ("sustainable urban mobility plans" (SUMP)), continuous public participation plays a major role [7].
Recent experiences have shown that public participation is not a universal remedy that solves contentious conflicts. For example, in conflicts involving conservation areas or private property, a solution might involve more than public participation. Nonetheless, public participation can help to work towards a concrete solution. Public participation can increase the accpetance of transportation planning projects, which in turn can improve the final outcome of project. For successful public participation, transportation planners will have to improve their skills at preparing and implementating public participation sessions. Assistance from professional moderators might be necessary as well.
In summary, public participation is an expression of culture. It describes the process of interaction between state and citizen. Correspondingly, the experiences made in Germany cannot be identically adapted to other countries. An international exchange of experiences and best practices can help to accelerate the learning process for all stakeholders.

References

[1] Brettschneider, F. & Schuster, W.: „Stuttgart 21“: Ein Großprojekt zwischen Protest und Akzeptanz, Stuttgart 2013

[2] Bundesministerium für Verkehr, Bau und Stadtentwicklung: Handbuch für eine gute Bürgerbeteiligung, Berlin, 2012

[3] Friedrich-Ebert-Stiftung, Abteilung Wirtschafts- und Sozialpolitik: Frühzeitige Bürgerbeteiligung für eine effizientere Verkehrsinfrastrukturplanung, Bonn, 2012

[4] Forschungsgesellschaft für Straßen – und Verkehrswesen (FGSV): Hinweise zur Beteiligung und Kooperation in der Verkehrsplanung, Köln, 2012

[5] Freie Hansestadt Bremen, Senator für Umwelt, Bau und Verkehr: Auswertung Online-Dialog Bremen bewegen, Stufe 1. http://bremenbewegen.de/wp-content/uploads/2013/06/bremendbewegung_st1web.pdf, 24.02.2014

[6] Roland Berger Strategy Consultants: Best-Practices-Studie zur Verkehrsinfrastrukturplanung und -finanzierung in der EU, Berlin 2013

[7] Wefering, F., Rupprecht, S., Bührmann, S. & Böhler-Baedeker, S.: Guidelines. Developing and Implementing a Sustainable Urban Mobility Plan, Köln 2014

THE IMPACT OF PUBLIC TRANSPORT PERFORMANCE IMPROVEMENTS ON SUSTAINABLE URBAN MOBILITY – AN EXAMPLE OF THE CITY OF ZAGREB

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Abstract

The global urbanisation process in the world is considering urban mobility as its top priority. Therefore, the public transport becomes more significant because of its spatial, energetic, economic and environmental advantages over other modes of transport and thus becomes the framework of sustainable mobility in most cities. Furthermore, the strengthening of public transportation in urban areas produces additional requirements in terms of its efficiency and operability. This is especially related to public surface transport (trams and buses) which is not segregated from other traffic, so the public transport process is constantly interfering with other vehicles sharing the same urban network. Consequently, this influence of other vehicles leads to commercial speed reductions in public transport and thus the attractiveness of public transport becomes significantly lower. The previous research on this subject has concluded that the yellow lane enforcement (as part of public transport priority by legislation) can lead to better transport process (travel times, network occupancy, and reliability) and other positive effects (energy savings, impact on the environment, reduction of external costs) at the entire urban network. This paper presents an analysis of priority by legislation for the entire tram network in the City of Zagreb – the main attention is dedicated to yellow lane contravention in the peak hour. The analysis of the research conducted in this paper, related to operational features and shared space characteristics, provided conclusions regarding the significance of interferences and possible improvements in both efficiency and effectiveness of tram network in the City of Zagreb.

Keywords: tram traffic, tram priority, network speed, City of Zagreb

1 Introduction

The basic principle of sustainable urban mobility is the modal shift of city trips in favour of public transport. Therefore, the public transport has to operate at peak efficiency in order to attract as many as passengers possible. There are two basic parameters of public transport efficiency: dwelling time at stops and running time along station spacings. Both parameters have impact on the operating speed at individual public transit lines and the network commercial speed associated with quality of service. Desirable running times can be achieved by applying public transport priority, in form of:

· priority by segregation;
· priority by legislation;
· signal priority.

1.1 The background of the research

Public transport priority is a subject of much research because it plays a significant part in quality of service. In recent times, different kinds of solutions related to priority have been developed in order to increase performance of public transport network. Pyrgidis & Chatziparaskeva (2012) studied signal priority of tram network in Athens and found that it is possible to expect increases in commercial speed between 15 % and 25 % [2]. Within the Civitas project family, many measures related to public transport have been implemented. Such of the measures are: new traffic lights and bus priority in Malmö, bus priority in Prague, public transport priority in Ljubljana, Rotterdam, Kraków and Suceava, bus rapid transit corridors in Toulouse, Lille and San Sebastián, new traffic light regulation in Vitoria-Gasteiz, yellow lane surveillance in Perugia and high-mobility corridor in Genova. All the measures resulted in commercial speed increases, travel time reductions and eventually, passenger satisfaction linked to modal shift [3].

The Civitas-Elan project was carried out in Zagreb, with one of the measures named „Giving priority to public transport". Signal priority was implemented at three intersections in Savska Street, resulting in 5 % time saving for the entire corridor [4]. In addition, a pilot-project was carried out which involved yellow lane enforcement by traffic police at the same corridor. Significant time saving of 25 % throughout the corridor was achieved [1].

The analysis of yellow lane priority on a single tram line (line number 4) was conducted by Brčić, Slavulj & Šojat (2012). The data was collected by GPS logging units placed in trams. A nonlinear model was developed for the optimisation purposes, with minimisation of the number of transport units as the objective function and constraints derived from network limitations. By using the Solver (MS Excel) and Nonlinear Programming (WinQSB) modules, the optimisation resulted in commercial speed increase of 8 % and savings of one transport unit, which is 7 % of the total number of transport units on the line [5].

1.2 Tram priority in the City of Zagreb

Tram network in Zagreb is a part of the urban road network, on which:

- 53 % is segregated completely (green lanes);
- 21 % is in separate road lanes (yellow lanes);
- 26 % is shared with other modes of transport (white lanes).

Commercial speed of tram network in the City of Zagreb is has been declining from 15.4 km h^{-1} (in 1999) to 13.0 km h^{-1} (in 2009), which is approximately 16 % (Fig. 1). This decline is the result of the increasing usage of urban road network by private cars. Besides the private cars freely using the white lanes, an additional problem is created by yellow lane contravention, because there is not any kind of yellow lane enforcement present in peak hour. The increasing usage of urban road network by private cars is also the reason for shorter green light percentages for trams at intersections. All these factors negatively influence the commercial speed of tram network in the City of Zagreb.

In the scope of priority by legislation, this paper presents a case study of the City of Zagreb and in particular, investigates what kind of changes would happen on tram network if yellow lane contravention was eliminated. This is done for peak hour because the probability of yellow lane contravention is highest for that period.

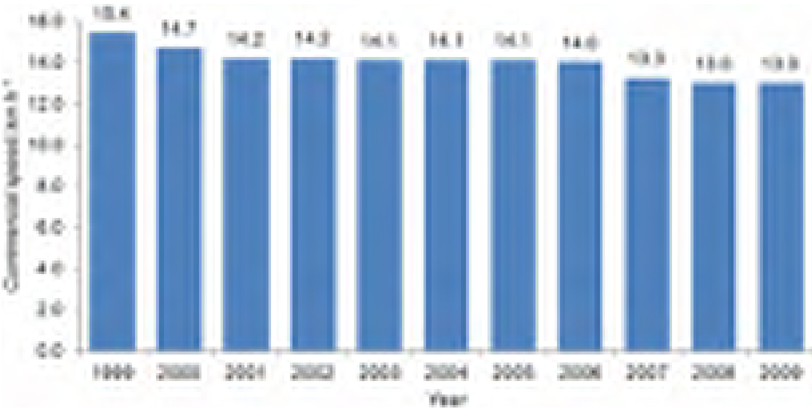

Figure 1 Commercial speed of tram network in the City of Zagreb [1]

2 Methodology

The input data was obtained from Zagreb Electric Tram, which is the public transport service provider in Zagreb [6]. For each transit line (l) the following data was available for the peak hour:
- capacity of transport units by type;
- number of transport units by type;
- operating speed ($V_{o,l}$);
- total line length ($L_{T,l}$);
- total length of yellow lanes ($L_{Y,l}$).

From the data above the following can be derived:
- addition of the capacities of transport units by type gives the line capacity (C_l);
- addition of line capacities gives the network capacity;
- addition of the number of transport units by type gives the number of transport units on a line (N_l).

The network commercial speed (V) is the average speed of the entire set of transport units on the network:

$$V = \frac{\Sigma_l V_{o,l} N_l}{\Sigma_l N_l} \tag{1}$$

The yellow lane percentage on a transit line (p_l) is the ratio of total length of yellow lanes and total line length:

$$p_l = \frac{L_{Y,l}}{L_{T,l}} \tag{2}$$

The operating capacity on a transit line (Q_l) can be derived from the fundamental equation of transport process in public transport network [7]:

$$Q_l = \frac{V_{o,l} C_l}{L_{T,l}} \tag{3}$$

The research methodology in this paper is based on the results in Brčić, Slavulj & Šojat (2012), in which the tram line 4, with the percentage p_4 produced increase in operational speed of i_4. A simple linear model will be used to estimate the increases in operational speed (i_l) for each other transit line by knowing their yellow lane percentages (p_l):

$$i_l = \frac{i_4}{p_4} p_l \tag{4}$$

The assumption that justifies the proposed model is the fact that the transit line 4 has the highest percentage of yellow lanes in the network and, in addition, the line operates throughout the key sections of the city. This makes the line number 4 a representative for yellow lane contravention, so authors presume that exactly this line would produce the most credible linear model. From the increase, new operational speeds for each transit line ($v_{o,l}$) can be derived:

$$v_{o,l} = (1 + i_l) V_{o,l} \tag{5}$$

Based on the model, two different scenarios are being considered by authors:
- Scenario 1 – if the number of transport units on each transit line remains unchanged, the new network commercial speed (v) as the indicator of quality of service for passengers is sought the same way as in the Eq. (1);
- Scenario 2 – if the operating capacity on each transit line remains unchanged, the new network capacity (c) as the indicator of savings for the operator is sought the same way as in the Eq. (3).

3 Research results

The rolling stock of Zagreb Electric Tram is shown in Table 1 – there are currently seven different types of transport units by capacity.

Table 1 The rolling stock of Zagreb Electric Tram [6]

Designation	Name	Year	Capacity
/	/	/	**sps veh^{-1}**
201+T	TMK 201 with 591	1974	213
401	ČKD Tatra T4	1977	103
401+T	ČKD Tatra T4 with B4	1977	217
301	ČKD Tatra KT4	1985	150
2100	TMK 2100	1994	242
2200	NT 2200	2005	202
2300	NT 2300	2009	130

Table 2 shows the layout of each type of transport unit on each transit line, and also the total number of units on each line in transport units (veh) as well as the line capacity in spaces (sps). This gives the network capacity of 34,872 sps made by 179 veh operating in peak hour.

Table 2 Tram rolling stock matrix [6]

Line	201+T	401	401+T	301	2100	2200	2300	N_l	C_l
/	veh	veh	veh	veh	veh	veh	veh	veh	sps
1				5				5	750
2	3					11		14	2861
3				8				8	1200
4			4			10		14	2888
5					8	4		12	2744
6			4			12		16	3292
7			4			12		16	3292
8			5					5	1085
9				9			1	10	1480
11						18		18	3636
12						15		15	3030
13				9			1	10	1480
14			4			12		16	3292
15		2						2	206
17						18		18	3636

Table 3 shows the input parameters for each transit line in order to get yellow lane percentage in percents (%) and operating capacity in spaces per hour (sps h^{-1}). Using Eq. (1), the network commercial speed in 2012 becomes 12.4 km h^{-1}, which is an even higher decline of 20 % compared to 1999 (the decline of network commercial speed was 16 % in 2009 compared to 1999).

Table 3 Parameters of tram network prior to implementation of the linear model

Line	Name	$V_{o,l}$	$L_{T,l}$	$L_{Y,l}$	p_l	Q_l
/	/	km h^{-1}	m	m	%	sps h^{-1}
1	Zapadni kolodvor – Borongaj	11.5	12022	1501	12	714
2	Črnomerec – Savišće	12.4	21784	4686	22	1625
3	Ljubljanica – Savišće	12.7	20766	3022	15	735
4	Savski most – Dubec	11.7	25186	16014	64	1337
5	Prečko – Kvaternikov trg	11.2	25182	2989	12	1224
6	Črnomerec – Sopot	13.1	20820	3584	17	2070
7	Savski most – Dubrava	13.6	26108	6655	25	1716
8	Mihaljevac – Zapruđe	13.2	16768	1055	6	857
9	Ljubljanica – Borongaj	10.8	14754	4936	33	1084
11	Črnomerec – Dubec	12.9	23978	13421	56	1949
12	Ljubljanica – Dubrava	11.7	18682	10418	56	1893
13	Žitnjak – Kvaternikov trg	11.5	22736	3918	17	746
14	Mihaljevac – Zapruđe	14.1	25650	3688	14	1806
15	Mihaljevac – Dolje	14.8	5422	0	0	562
17	Prečko – Borongaj	11.8	25356	3900	15	1694

After the implementation of the simple linear model, the results for each transit line are shown in Table 4 as increases in operational speed (%), new operational speeds in kilometres per hour (km h^{-1}) and new capacities (sps). The constant in Eq. (4) is 0.127, which means that for every percent of yellow lanes on the line there will be about one eight of percent of increase in operational speed.

Table 4 Parameters of tram network after the linear model

Line	Name	i_l	$v_{o,l}$	c_l
/	/	%	km h^{-1}	sps
1	Zapadni kolodvor – Borongaj	1.6	11.6	738
2	Črnomerec – Savišće	2.7	12.7	2785
3	Ljubljanica – Savišće	1.8	13.0	1178
4	Savski most – Dubec	8.1	12.6	2672
5	Prečko – Kvaternikov trg	1.5	11.4	2703
6	Črnomerec – Sopot	2.2	13.4	3222
7	Savski most – Dubrava	3.2	14.1	3189
8	Mihaljevac – Zapruđe	0.8	13.3	1076
9	Ljubljanica – Borongaj	4.2	11.3	1420
11	Črnomerec – Dubec	7.1	13.8	3395
12	Ljubljanica – Dubrava	7.1	12.5	2830
13	Žitnjak – Kvaternikov trg	2.2	11.7	1448
14	Mihaljevac – Zapruđe	1.8	14.3	3233
15	Mihaljevac – Dolje	0.0	14.8	206
17	Prečko – Borongaj	2.0	12.0	3566

The final results for each scenario are the following:
- In scenario 1, the network commercial speed for the same number of transport units rose to 12.8 km h^{-1}, which is an increase of 3.6 %;
- In scenario 2, the network capacity for the same operating capacity dropped to 33,662 sps, which is a decline of 1,210 sps, or 3.5 %.

The savings in network capacity can be imagined as the removal of 6.0 veh of 2200, 5.0 veh of 2100, or the entire transit line number 3.

4 Discussion

In the scope of the two scenarios presented in this paper, the linear model has its drawbacks. Firstly, the probability of yellow lane contravention is extremely complex because it depends on various factors. Such factors are traffic flows next to yellow lanes, pedestrians at crossings, etc. In addition, yellow lane contravention shows different patterns in different parts of tram network because it is subject to the cumulative effect. Therefore, the linear model used to describe yellow lane contravention on other transit lanes can only serve as an approximation. In reality, it is necessary to develop an empirical model which would provide the data for each part of the network separately.
Another drawback is the assumption that it is possible to establish relationship between yellow lane contravention and operating speed while in reality that relationship is only indirect. The yellow lane contravention has impact solely on the running times (it has no influence on dwelling times at stops whatsoever). The justification in this case is the lack of detail in the available data. The data should contain running times for the entire tram network, and even more precise, running times for every section by type of segregation as in the methodology presented in Brčić, Slavulj & Šojat (2012).
The transportation process used in this paper is also simplified, so it is very roughly presented when considering terminal times (linearity assumed) or critical volume-to-capacity ratios of transport units (100 % assumed). Those elements play a very important part in the optimisation model, so they have to be taken into account.

From the passenger perspective, the 3.6 % increase in network commercial speed is not very significant, but if yellow lane contravention was eliminated, the quality of service would sharply increase because large oscillations of travel times would also be eliminated, so passengers would not have to input time surplus for the worst case scenario trip, and by this their travel time could be expected to become smaller anyway. Even small increases of network commercial speed can significantly reduce external costs of travel.
Although the savings in network capacity are 3.5 %, they can also be significant because the removal of transport units results in considerate reductions of personnel costs, vehicle operating costs, vehicle maintenance costs, network congestion and the probability of disruptions in power grid even for a small number of units.

5 Conclusions and suggestions

The research conducted in this paper showed that possible elimination of yellow lane contravention (which can be achieved by yellow lane enforcement) brings significant improvements in tram network, interpreted as network commercial speed increase (3.6 %) or network capacity decrease (3.5 %). The benefits are cost reductions and better quality of service. Based on previous experiences and the ones obtained in this paper, if every kind of priority was combined together, authors estimate that network commercial speed in the City of Zagreb could increase up to 30%.
Transport process in this paper was simplified to adapt to the input data. However, the mutual interaction of elements and constraints from the optimisation process itself require thinking of the optimisation process as a nonlinear problem, which has to be solved by appropriate software tools. For such a problem, the input data has to have sufficient quality and the amount of detail.
To ensure better quality of the input data, a geo-referenced network with all the necessary features and with the possibility to change in time is recommended. Also, the tram priority can sometimes leave the negative impact on the road network, so the only way to estimate the consequences is the usage of simulation tools. Only in such manner tram network can be successfully maintained from the traffic technology perspective.
In the near future, it is necessary to fully examine the dynamic efficiency of tram network in the City of Zagreb (to include all possible elements influencing network commercial speed). Such a study would reveal the weak points on the network (points with the lowest capacity), and appropriate corrections done on those points would result in better network commercial speed and consequently, better quality of service for passengers.

References

[1] Javni gradski prijevoz kao element održive mobilnosti u Gradu Zagrebu, www.hak.hr, 06.02.2014.

[2] Pyrgidis, C. & Chatziparaskeva, M.: The impact of the implementation of green wave in the traffic light system of a tramway line – the case of Athens Tramway, 2nd International Conference on Road and Rail Infrastructure – CETRA 2012, 7–9 May Dubrovnik, Croatia, pp. 891-898.

[3] Slavulj, M., Brčić, D. & Ćosić, M.: Improvement of urban public transport with transportation demand management, Suvremeni promet : časopis za pitanja teorije i prakse prometa, 32, pp. 355-359, 2012.

[4] Dodjela prioriteta tramvajima u Gradu Zagrebu, www.civitaszagreb.hr, 06.02.2014.

[5] Brčić, D., Slavulj, M. & Šojat, D.: Analysis of tram priority in the City of Zagreb, 32nd Conference on Transportation Systems with International Participation, Zagreb, pp. 173-176, 2012.

[6] Zagrebački električni tramvaj, www.zet.hr, 06.02.2014.

[7] Vuchic, V. R.: Urban Transit Operations, Planning and Economics, John Wiley and Sons, Inc., 2005.

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EVALUATION OF THE VARIABLE MESSAGE SIGNS (VMS) SYSTEM IN THE CENTRAL AREA OF THESSALONIKI FROM THE USER POINT OF VIEW

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Abstract

In the framework of this paper, VMS systems in two arterial streets in the central area of the city of Thessaloniki, Greece, are evaluated from the user point of view. Evaluation is based on a questionnaire-based survey which took place in Thessaloniki, during September 2013. A total number of 167 questionnaires addressed to drivers who use the under study road segments were collected and analysed. According to the results of the survey, drivers are generally in favour of the VMS system but they ask for different types of information to be displayed apart from the travel time. It was found that “time” is considered to be the most important factor for the drivers in order to choose their route, compared to “cost” and “comfort”. In addition, the majority of the drivers do not have confidence in the VMS message. It seems that a substantial part of the drivers continue their trip in case of a traffic congestion message.

Keywords: Variable Message Signs, evaluation, driver information, traffic

1 Introduction

Variable Message Signs (VMS) is an application of changeable traffic labelling systems that play a significant role in traffic control and management of road networks in many countries all over the world. VMS provide the drivers with pre-defined and real time information and are usually installed on the side or above the road. The content of the messages displayed, varies. It refers to the current traffic and weather conditions, the location of a traffic accident, special events (i.e. cultural or sports events) in the nearby area, travel time, alternative routes in case of congestion, etc. An important study about the evaluation of VMS on the highway network (e.g., impact on traffic flow, road safety, environment) was conducted for the Slovenian part of Corridor V [1]. In the framework of an evaluation concerning ITS applications in the VIKING area i.e. Finland, Sweden, Norway, Denmark and Northern Germany various VMS applications were also examined [2]. It must be noticed that accuracy and relevance of the information provided by the VMS is an important factor for their success [3]. The issues of VMS harmonisation and interoperability are also very important and they have been addressed in a report by CEDR’s Task Group O9 [4]. Finally, the environmental impacts of VMS are also important [5].
In the framework of this paper, VMS systems in two arterial streets (Queen Olga, Konstantinos Karamanlis) in the central area of the city of Thessaloniki, Greece, are evaluated from the user point of view. It must be mentioned at this point that the two VMS under investigation are located at strategic points of the road network in order to inform drivers who are directed towards the city centre. They usually provide information about travel time for specific destinations in the city centre. Evaluation is based on a questionnaire-based survey which took place in Thessaloniki, during September 2013 in the framework of the research activities of the Faculty of Rural & Surveying Engineering of the Aristotle University of Thessaloniki [6]. A total

number of 167 questionnaires addressed to drivers who use the under study road segments were collected and analysed.

2 Design of the questionnaire

For the purposes of the specific research a questionnaire was designed. This questionnaire was addressed to the drivers who use the road network where the VMS are installed. The first part of the questionnaire refers to the socio-economic characteristics of the interviewee as follows:
· Gender
· Age: 18-30, 31-45, 46-60 and >60 years old;
· Occupation: a) employee in the public sector, b) student, c) homemaker, d) self-employed worker, e) employee in the private sector, f) pensioner, g) other;
· Household: a) number of members, b) number of private cars.

Questions about the income and education level were avoided because of the possibility to obtain wrong answers. The second part of the questionnaire refers to the trip of the interviewee as follows:
· Number of years driving: 1-2, 3-5, 6-10, 11-15, 16-25, >25;
· Number of trips made through the city center during a typical weekday (i.e. Monday to Friday) with a private car as a driver or as a passenger: a) once a day, b) 2-3 times per day, c) > 3 times a day, d) 1-2 times per week, e) 3-4 times per week, f) 1-2 times per month;
· Number of trips during Saturday and during Sunday;
· Common trip purpose towards the city center: a) work, b) trade activities, c) recreation activities, d) other.
· Main road corridor used to access city centre & trip origin (street address, area);
· Factors considered being the most important for route selection: a) cost, b) time, c) comfort (interviewees had to choose one of the following answers as far as each one of the three factors are concerned: a) not at all, b) not much, c) enough, d) a lot, e) very much).

The third part of the questionnaire refers to the utilization of the VMS and the recognition of the information provided as something which is significant:
· Observation of the VMS: a) yes, b) no;
· Degree of utilization of the information provided in case of positive answer in the previous question: a) not at all, b) not much, c) a lot;
· Confidence for the information provided: a) not at all, b) not much, c) a lot, d) very much;
· Impact to decision taking because of the VMS (e.g. alternative route): a) not at all, b) not much, c) enough, d) a lot, e) very much;
· Possible action in case of important message concerning traffic congestion: a) continue my trip without any change, b) change route, c) change transport mode (e.g., from private car to public transport, taxi etc.), d) cancel trip, e) other.
· Belief that VMS can contribute to the reduction of traffic congestion: a) not at all, b) not much, c) enough, d) a lot, e) very much;
· Type of information that is considered to be necessary to appear in the VMS (use of scale 1-5 for the answers): Traffic conditions, Weather conditions, Traffic accidents, Location of traffic accidents, Possible road hazards, Work zones, Duration of travel, Possible alternative routes, Information for cultural events, Emergency situations.

3 Analysis of results

Hereinafter the analysis of the results of the questionnaire-based survey is presented with the use of descriptive statistics. As far as "gender" is concerned, 104 (62.3%) of the respondents are men and 63 (37.7%) are women. The age distribution of the respondents is presented in Table 1.

Table 1 Age distribution of the respondents

Age	Frequency	Percent	Cumulative percent
18-30	78	46.7	46.7
31-45	49	29.3	76.0
46-60	32	19.2	95.2
>60	8	4.8	100.0

As presented in Table 2, the vast majority of the respondents are between 18 and 30 years old, something which can be explained as follows: either young people are characterized by increased mobility levels or they are more eager to take part to questionnaire-based surveys or both reasons exist at the same time. As far as the occupation of the respondents is concerned, "homemaker" and "pensioner" appear to have the lowest values. "Driving experience" is presented in Table 2.

Table 2 Driving experience of the respondents

Years of driving	Frequency	Percent	Cumulative Percent
<1	3	1.8	1.8
1-2	16	9.6	11.4
3-5	32	19.2	30.5
6-10	29	17.4	47.9
11-15	36	21.6	69.5
16-25	17	10.2	79.6
>25	34	20.4	100

As presented in Table 2, more than 50% of the respondents can be considered as "highly" experienced drivers (with more than eleven years of driving). Results concerning "number of cars" are presented in Table 3.

Table 3 Number of cars in the households of the respondents

Cars	Frequency	Percent	Cumulative Percent
0	4	2.4	2.4
1	69	41.3	43.7
2	60	35.9	79.6
3	22	13.2	92.8
4	10	6.0	98.8
5	2	1.2	100.0
Total	167	100	–

More than 77% of the respondents stated that they have one or two cars in their household. It is also found, as expected, that the larger the size of the household, the higher the number of cars they have. The number of trips made through the city center is presented in Table 4.

Table 4 Number of trips made through the city center during a typical weekday

Trips	Frequency	Percent	Cumulative percent
One /day	37	22.2	22.2
2-3 /day	35	21.0	43.1
› 3 /day	11	6.6	49.7
1-2 /week	39	23.4	73.1
3-4 /week	27	16.2	89.2
1-2 /month	18	10.8	100.0

As presented in Table 4, almost half of the respondents make more than one trip through the city center, and thus it can be considered that they are familiar with the road network of the specific area. The results concerning "trip purpose" of the respondents are presented in Table 5.

Table 5 Trip purpose of the respondents

Trip purpose	Percent
Work	33.53
Trade activities	5.99
Recreation activities	35.93
Other	2.40
Combination of reasons	22.15

More than one third of the trips have "recreation activities" as purpose. This can be explained due to the fact that the city center is characterized by a large number of land uses related to recreation. The factors affecting drivers' choice about their route are presented in Table 6.
As presented in Table 6, "Time" is considered to be the most important factor for the drivers in order to choose their route, compared to "cost" and "comfort". It is expected that some drivers do not note that there are VMS in the road network. It is also known that not all drivers show confidence in the VMS message. The combined results about these issues are presented in Table 7.

Table 6 Factors affecting the route choice

	Frequency	Percent	Cumulative Percent
	Cost		
Not at all	19	11.4	11.4
Not much	44	26.3	37.7
Enough	40	24.0	61.7
A lot	26	15.6	77.2
Very much	38	22.8	100.0
	Time		
Not at all	2	1.2	1.2
Not much	10	6.0	7.2
Enough	26	15.6	22.8
A lot	50	29.9	52.7
Very much	79	47.3	100.0

Table 6 Factors affecting the route choice (continued)

	Comfort		
Not at all	18	10.8	10.8
Not much	30	18.0	28.7
Enough	54	32.3	61.1
A lot	34	20.4	81.4
Very much	31	18.6	100.0

Table 7 Level of confidence in the VMS message and VMS observation

	Level of confidence for the message			
	Not at all	Not much	A lot	Very much
Drivers who observe VMS in the road network	16	87	36	3
Drivers who do not observe VMS in the road network	22	3	0	0

As presented in Table 7, the majority of the drivers do not have confidence in the VMS message. The impact to decision taking because of the VMS message (traffic congestion) is presented in Table 8.

Table 8 Impact of VMS traffic congestion message to decision taking by the drivers

Possible action	Impact				
	Not at all	Not much	Enough	A lot	Very much
Continue my trip without any change	16	12	2	3	0
Change route	41	33	35	16	4
Change transport mode	0	0	1	0	0
Cancel trip	3	0	0	0	0
Other	0	1	0	0	0

It seems that the vast majority of the drivers either continue their trip or change their route in case of a traffic congestion message. Only a few drivers would prefer to change their transport mode or to cancel their trip. Table 9 presents the type of information that is considered to be necessary to appear in the VMS (mean values, scale 1-5).

Table 9 Type of information proposed to be presented in the VMS

Type of information	Mean value (scale 1-5)
Traffic conditions	4.13
Weather conditions	2.81
Traffic accidents	4.35
Location of traffic accidents	4.28
Possible road hazards	3.86
Work zones	3.98
Duration of travel	3.83
Possible alternative routes	3.57
Information for cultural events	2.30
Emergency situations	4.20

As presented in Table 9, information about traffic conditions, traffic accidents in general and also specific information (e.g., location of an accident) and emergency situations are considered by the drivers as the most valuable types of information.

4 Conclusions

Drivers have the tendency to question the displayed information in cases where this information remains the same for long time periods. If the displayed travel time is not accompanied by the cause of an event, drivers do not change their travel behaviour. The location of the VMS installations is considered by the researchers to be a very critical factor for the success of the system. According to the results of the survey, drivers are generally in favour of the VMS system but they ask for different types of information to be displayed apart from the travel time. The majority of the drivers have enough experience in driving and they are also familiar with the under study road network. “Time” is considered to be the most important factor for the drivers in order to choose their route, compared to “cost” and “comfort”. In addition, the majority of the drivers do not have confidence in the VMS message. It seems that a substantial part of the drivers continue their trip in case of a traffic congestion message. In any case, these results can be used only as an indication of the users’ perception about the usefulness of the VMS system. Extensive questionnaire-based surveys which must be repeated over time are necessary to obtain more robust results.

References

[1] Merlin, B. & Vindišar, B.: Evaluation of VMS deployment on Slovenian part of corridor V, European Commission, DG-TREN, TEMPO Secretariat, 2009.

[2] Dörge, L., Kulmala, R., Lind G., Rydmell, C., Wold, H. & Sage, D.: Euro-Regional Project VIKING – MIP, VIKING Overview of evaluations, MIP2005, 2006.

[3] Nygardhs, S.: Literature review on variable message signs, (VMS) 2006–2009, VTI notat 15A-2011, 2011.

[4] CEDR’s Task Group O9: VMS harmonization in Europe, CEDRE, 2009.

[5] Basbas, S.: Environmental impacts from the operation of variable message signs in the road network, Journal of Environmental Protection and Ecology, Vol.8, No.1, pp.204-211, 2007.

[6] Betos, A., Kyriazopoulos, D. & Nikolaidis M.: Evaluation of the VMS system in the central area of the city of Thessaloniki, Diploma Thesis, Faculty of Rural & Surveying Engineering, Aristotle University of Thessaloniki, 2013.

TESTING A MIXTURE MODEL FOR THE DISTRIBUTION OF ARRIVAL TIME OF URBAN RAILWAY TRAVELLERS

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Abstract

Recently the evaluation of the travel time reliability has become great concern in transportation planning and many studies have investigated the travel time reliability. However most of them have focused on the reliability regarding road transports. Then the reliability of rail transport is focused on in this study. Authors have already investigated the departure time decision of railway users in Tokyo Metropolitan area (TMA) and developed a model describing a spare time for not being late behind the desired arrival time by considering the heterogeneity of the distribution of the late arrival. In order to estimate the coefficients of the model, we set a strong condition that the railway users in TMA recognized that the delay of arrival time obeyed exponential distribution. However the difference between the desired arrival time and the actual arrival time does not always obey exponential distribution. Then, we started the research to ease this strong condition by adopting the mixture distribution model. There are few studies investigating departure time decision dealing the mixture model. An internet survey was conducted to collect the data for the study. The question items are name of origin and destination stations, travel path, departure time, desired arrival time, required travel time, distribution of arrival time and socio-economic attributes.

Keywords: urban railway, reliability of travel time, mixture distribution model

1 Introduction

Reliability of travel time is one of the important factors affecting mode choice and route choice behaviour. Recently, many studies have investigated variability of travel time and engaged in developing methodology to evaluate economic value of the travel time reliability. Most of them have focused on the reliability of road traffic. Meanwhile, research on the travel time reliability of railroad and air transport which operates based on the planned timetable has not sufficiently advanced so far.
We have investigated the travel time reliability of the railroad from the point of departure time decision of railroad users [1-4]. As the results of our previous studies, it becomes clear that railroad user prepares buffer time to deal with the delay of railroad. Furthermore, it indicates that the buffer time is influenced by several factors such as travel distance, usages frequency of railroad, and number of times of transfer.
In order to analyze the departure time choice behavior, we set a strong condition that the railway users recognized that the delay of arrival time obeyed exponential distribution. However the difference between the desired arrival time and the actual arrival time does not always obey exponential distribution. Then, we got to work on research to ease this strong condition by adopting the mixture distribution model. There are few studies investigating departure

time decision dealing the mixture model. The contents of this paper is as follows. In chapter 2, the data used for the analysis is descrived. In the chapter 3, the charactoristics of the difference between the desired and realized arrival times is examined. In chapter 4, Gaussian Mixture Model (GMM) is applied to estimate the probability density function of the difference between the desired and realized arrival times. In chapter 6, the results of this paper is summarized.

2 DATA

A web survey utilizing the internet was conducted in this study. The respondents were monitors of the Macromill, INC. summary of the survey was shown in table 1.

Table 1 Summary of web survey.

Conditions of the respondents	Resident area: Tokyo, Kanagawa, Saitama, and Chiba prefectures Age: over 15 years old Occupation: Yes Other condition: person who uses railway more than 5 days in a week
Date	2012.March,26-28
Number of respondents	1700
Valid respondents	1418

2.1 Measurement of accuracy of arrival time

Table 2 shows the question about desired arrival time and Table 3 shows the question about the arrival situation. Data of the frequency of difference from the desired arrival time was produced by these questions. Data format is shown as table 4.

Table 2 Inquiries about desired arrival time

Question 1.	What time is the most desired arrival time at the station of the destination?

Table 3 Inquiries about the arrival situation

Question 2-1.	How many times do you arrive at the station within two minutes difference from the desired arrival time?
	Answer: ( ) time(s) out of ten times
Question 2-2.	How many times do you arrive at the station more than 2 minutes before the desired arrival time?
	Answer: ( ) time(s) out of ten times
Question 2-3.	How many times do you arrive at the station more than 2 minutes after the desired arrival time?
	Answer: ( ) time(s) out of ten times
Question 3-1.	Your answer of the Question 2-2 is ( ) time(s). Well then how many times do you arrive at the station more than 5 minutes before the desired arrival time?
Question 3-2.	Your answer of the Question 2-2 is ( ) time(s). Well then how many times do you arrive at the station more than 10 minutes before the desired arrival time?
Question 4-1.	Your answer of the Question 2-3 is ( ) time(s). Well then how many times do you arrive at the station more than 5 minutes after the desired arrival time?
Question 4-2.	Your answer of the Question 2-3 is ( ) time(s). Well then how many times do you arrive at the station more than 10 minutes after the desired arrival time?

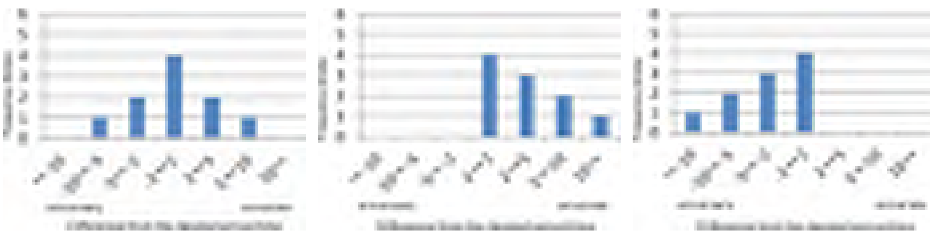
Table 4 Data format of arrival situation

ID	Early arrival				Late arrival		
	More than 10min.	5 min. to 10 min.	2 min. to 5 min.	Within 2 min.	2 min. to 5 min.	5 min. to 10 min.	More than 10min.
1	1	1	1	2	3	0	2
2	0	0	2	3	5	0	0
3	0	2	2	2	2	2	0
...	...	...	...	...	...	...	...
1313	0	0	1	8	1	0	0
1414	0	0	0	4	3	2	1

3 Characteristics of the difference between the desired and realized arrival times

3.1 Patterns of the difference between the desired and realized arrival time

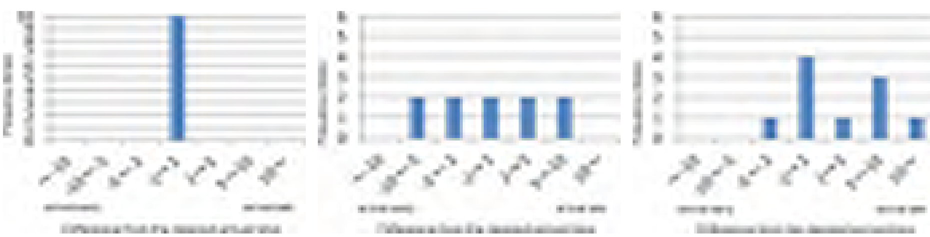
The difference between the desired and realized arrival time was analyzed. After examining the data regarding the difference, it became clear that there were six, from (a) to (g), patterns as shown in the Figure 1.



(a) symmetric pattern (449/1418 cases)

(b) late arrival pattern (434/1418 cases)

(c) early arrival pattern (41/1418 cases)



(d) on schedule pattern (333/1418 cases)

(f) Uniformly symmetric pattern (31/1418 cases)

(g) Nondesrict pattern (130/1418 cases)

Figure 1 Patters of the difference from the desired arraival time.

4 Methodology

4.1 Mixture distribution model

Mixture distribution is the distribution that mixed plural distribution. The probability density function of mixture distribution is written in (1).

$$p(x;\theta)=\sum_{m=1}^{M}\pi_m h(x;\omega_m). \tag{1}$$

where:

M number of mixture components;
$h\ (x;\omega_m)$ component distribution;
$\theta=\{(\pi_m,\omega_m),1\leq m\leq M\}$ parameter of distribution associated with component i, observation i;
π_m mixture weight and it satisfies $\sum_{m=1}^{M}\pi_m=1$.

In this study, Gaussian mixture distribution that is most general mixture distribution, is applied as the component distribution as shown in (2).

$$h(x;\theta)=\sum_{m=1}^{M}\pi_m\varphi(x;\mu_m,\Sigma_m). \tag{2}$$

where:

$\phi(x;\mu_m,\Sigma_m)$ Gaussian distribution;
μ_m mean of component distribution;
Σ_m variance of component distribution.

By estimating the maximum likelihood estimators of the all parameters, the shape of the mixture distribution is obtained. However the number of component distribution, M, is unknown so that the M is estimated by EM(Expectation-Maximization) algorithm.

4.2 Estimation results

The results of parameter estimation is described in this section. A table in the Figure 2 shows the estimated parameters such as weight, mean and variance. The number of components is four and each component is indicated by different colours. The graph shows the probability density function of each component. Furthermore, the probability density function of mixture distribution is shown by the black broken line.
Reproducibility of the estimated mixture distribution is confirmed. Figure 3 shows the result of comparison between the histogram of the observation and the estimated probability of mixture distribution. As seen in the figure, the reproducibility of the mixture model to the observation is high.

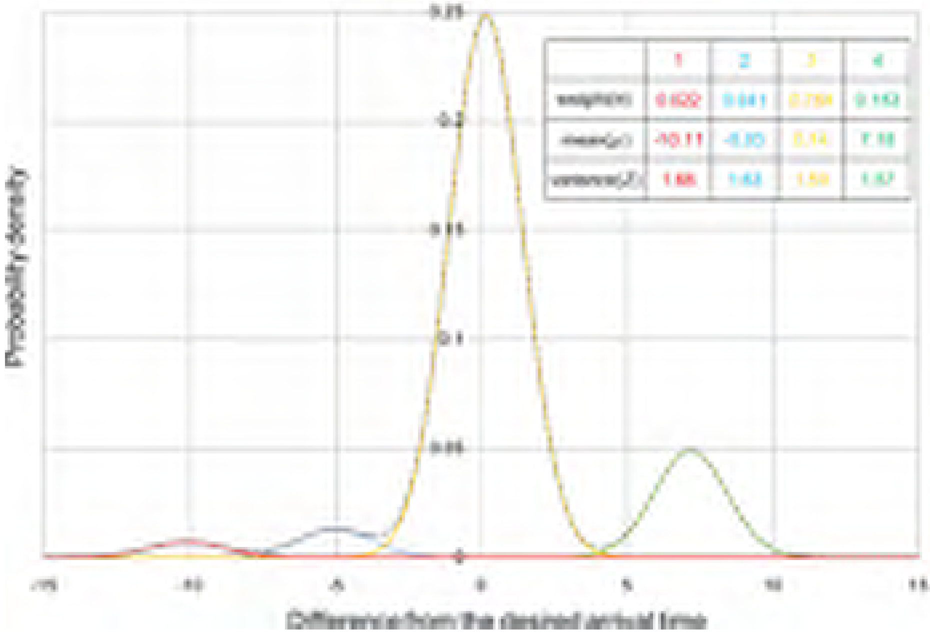


Figure 2 Density functions of component distributions and mixture distribution

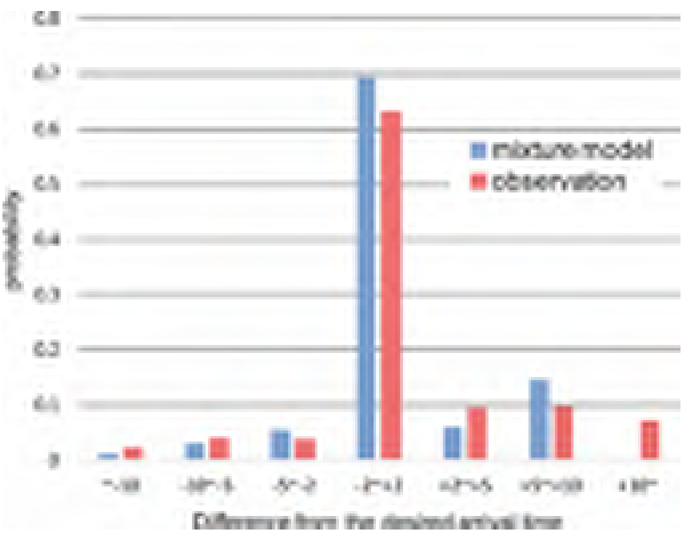


Figure 3 Reproducibility of the estimated mixture distribution

5 Conclusion

Our previous studies regarding departure time decision of railway users, there was a strong condition that the railway user recognized that the delay of arrival time obeyed exponential distribution. However the difference between the desired arrival time and the actual arrival time does not always obey exponential distribution. Then, we commenced the research for easing this strong condition by appling the mixture distribution model.
In this study, we confirmed the availability of the mixture model which can consider the arrival situation such as arrival on time, arrival early and arrival late. The estimated mixture distribution can explain the characteristics of the arrival situation.
In this study, Gaussian distribution was applied to all component distribution. However, as shown in the Figure 1, there are several patterns in arrival situation. To consider this diversity of the arrival situation, we study the availability of mixture model that applied different distribution to component distributions.

Reference

[1] Takada, K., Fujiu, M. & Yokoyama, S.: Perception of the Travel Time Reliability of the Urban Railway Service in Tokyo, Proceedings of the First International Conference on Road and Rail Infrastructures – CETRA 2010, pp. 873-879, 2010.

[2] Takada, K., Fujiu, M.: Study of Willingness to Pay for Reducing Lost Time of Railway Users, Journal of Eastern Asia Society for Transportation Studies, Vol. 8, 1464-1474, 2010.

[3] Takada,K.,Fujiu, M., Yokoyama, S. & Suzuki, T.: Departure Time Decision of Railway Users under Travel Time Uncertainty, Proceedings of the Eastern Asia Society for Transportation Studies, Vol. 8, pp. 209-219, 2011.

[4] Takada, K., Fujiu, M., Sugiyama, S.: Behavior Analysis of Departure Time Decision considering Redundancy of Railroad Network, Proceedings of the Second International Conference on Road and Rail Infrastructure – CETRA 2012, pp. 81-87, 2012.

ANALYSE OF THE ACCESSIBILITY OF PEOPLE WITH DISABILITIES OR REDUCED MOBILITY USING URBAN TRANSPORT TO HEALTH TREATMENT

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Abstract

The objective of this work is to analyse the accessibility of the people with disabilities or reduced mobility using urban transport for health treatment. The study was conducted in the city of Campinas – Brazil, considered as a center of excellence in health care with two renowned courses in medicine: one at the State University of Campinas – UNICAMP and Pontifical Catholic University of Campinas. For this purpose it was decided to analyse the stopping points of the public transport bus line 3:30 serving the Hospital of UNICAMP since this bus line has vehicles with equipment that allows access for people with mobility impairments. It was made a field survey georeferencing all bus stops and verifying for accessibility equipment, curbside cutouts, sidewalk conditions, tactile flooring, cover, bus schedule informations, etc. To perform the analysis was developed a Geographic Information System using MapInfo software (version 10.0) with the digital base provided by Campinas Municipal Development Company (EMDEC) which has as reference surface the Hayford ellipsoid, Córrego Alegre horizontal datum and Universal Transverse Mercator (UTM) coordinate system. After the final analysis it can be concluded that none of the line bus stops has all accessibility equipments focused on the field survey. It is important to mention that the relevance of this study is not only by the results but by the small number of studies that deal with this very important topic for social inclusion.

Keywords: accessibility, people with disabilities or reduced mobility, urban transport

1 Introduction

It can be argued that social inclusion is a measure leading to the drafting of laws and services aimed at meeting the special needs of persons with disabilities or reduced mobility. By creating equipment, laws and mechanisms to adapt the common social systems to these individuals difficulties, the social inclusion aims to transform society into a viable place for equal socializing among all people, regardless of their intellectual abilities, motor difficulties or potentialities. For social inclusion to be successful, you must associate them with the premises of Universal Design, which is characterized by the promotion of accessibility to all segments of the population through the creation of environments capable of harbouring the differences, unlike other space adaptation plans for people with disabilities who were characterized by unique and segregating environments [1].
Thus, this study aims to use the resources of a Geographic Information System to analyse the accessibility of persons with disabilities or reduced mobility using urban transport for health treatment.

2 Case study

The study was conducted in the city of Campinas – Brazil, considered as a center of excellence in health care with two renowned courses in medicine: one at the State University of Campinas – UNICAMP and Pontifical Catholic University of Campinas. For this purpose it was decided to analyse the stopping points of the public transport bus line 3:30 serving the Hospital of UNICAMP since this bus line has vehicles with equipment that allows access for people with mobility impairments. This route is approximately 33 km long. The course is conducted in 50 minutes approximately, and at peak times the service occurs every 8 minutes and in normal times every 15 minutes on average. It was made a field survey georeferencing all bus stops and verifying for accessibility equipment, curbside cutouts, sidewalk conditions, tactile flooring, cover, bus schedule informations etc.
Towards the Hospital of Unicamp (Figure 1) for the Central Terminal of Campinas (Figure 2) were analysed and 32 stopping points in the opposite direction, 30. Here are some photos taken at this stage (cf. Figures 3 – 8).

Figure 1 Clinics Hospital of the State University of Campinas

Figure 2 Campinas Central Bus Terminal

Figure 3 Examples of regularized sidewalks with curbside cutouts for easy access to people with reduced mobility, however, with the absence of elevated curbside to facilitate boarding of wheelchair users

Figure 4 Example of elevated curbside to facilitate boarding of wheelchair users

Figure 5 Obstruction in a walkway giving access to a stopping point located on Zeferino Vaz highway, shown in Figure 6

Figure 6 Stopping point at Zeferino Vaz highway

Figure 7 Stopping point fully prepared with regularized sidewalks

Figure 8 Ramps to facilitate access of persons with disabilities or reduced mobility

3 Development of GIS

To perform the analysis was developed a Geographic Information System [2] using MapInfo software (version 10.0) [3] with the digital base provided by Campinas Municipal Development Company (EMDEC) which has as reference surface the Hayford ellipsoid, Córrego Alegre horizontal datum and Universal Transverse Mercator (UTM) coordinate system [4]. The equipments were classified according to ISO 9050:2004 recommendations and the universal design concepts [5]. As a result was elaborated the stopping points classification thematic map shown in Figure 9.

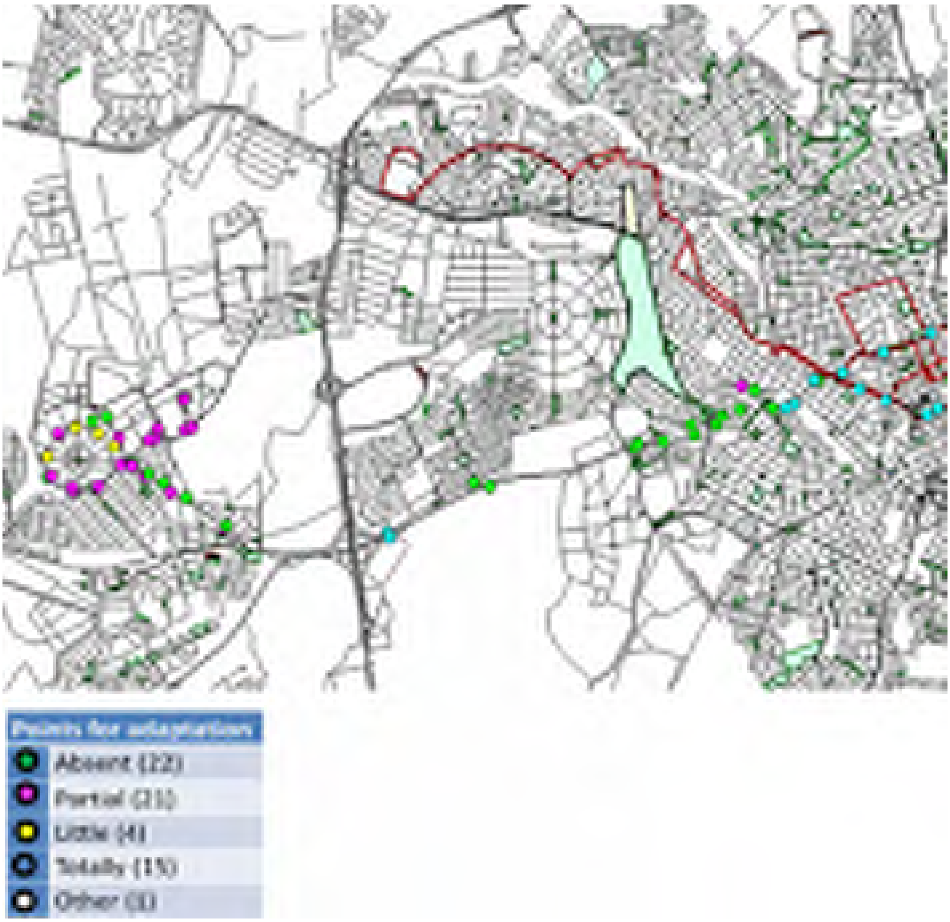

Figure 9 Stopping points classification thematic map

4 Conclusions

The study showed that are recurrent the contradictions that, historically, are presented about several ways of exclusion, manifested by prejudice, intolerance and segregation. Opposing to it, the urban transport is presented as a form to promote the social welfare because with it is possible to provide the access to health treatment.

The technical visits, taking as references NBR 9050 (ABNT 2004), pointed out that despite significant improvements enforced by specific laws that guarantee to the disabilities or people with reduced mobility the right to citizenship, the physical environment of such places still have many obstacles that impede their mobility, showing its unreadiness to receive them. It was also observed that the actions on accessibility have been marked by the adoption of palliative solutions that hinder the optimal use of space and reinforce segregation.
The GIS developed in this study represents a great advance because the data provide to the local government a tool of analyses interactive for visualization and query. Additionally, the broad coverage possible with the GIS shows the potential to generate a culture that benefits from adopting the principles of universal design to public transportation and Stopping points. After the final analysis it can be concluded that approximately 20% of the route stopping points have all equipment focused on the accessibility field survey. It is important to mention that the relevance of this study is not only the results but the small number of studies that deal with this very important topic for social inclusion.

References

[1] Duarte, C.R., Cohen, R.: Developing a method for creating a guide accessible to Rio de Janeiro, in: Proceedings of 2nd International Seminar on Contemporary Visions, pp. 1-13, 2004, http://www.proacesso.fau.ufrj.br/artigos/Metodologia_para_Guia_de_Acessibilidade__Visoes_Contemp.pdf (accessed June 7, 2013). (in Portuguese)

[2] Casanova, M. A., Camara, G., Davis Jr., C., Vinhas, L., Queiroz, G. R.: Geographical Databases, 2005. (in Portuguese)

[3] http:// http://www.mapinfo.com, 30.03.2013.

[4] Silva, D.L.N.: Using a geographic information system to improve transportation specifically for people with physical impairments, Master's Dissertation, Universidade Estadual de Campinas, 2004. (in Portuguese)

[5] ABNT. Brazilian Association of Technical Standards – NBR 9050 – Accessibility to buildings, furniture, equipment and urban spaces, 2004. (in Portuguese)

PROBLEMS IN PLANNING OF THE PRIMARY ROAD CORRIDORS IN THE CITIES ON THE EXAMPLE OF THE CITY OF ZAGREB

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Abstract

Primary road corridors are the main city road corridors for acceptance and distribution of traffic. This paper describes methodological approach to define the optimal solution for road corridors that can take the traffic load. Proves the need for upgrading the secondary road network along the primary road corridors based on the analysis of the transport corridor. Discusses calculated transport effects of alternative solutions and compares them with investments for the construction or reconstruction of the corridor. Displays the construction characteristics of road corridor with a series of split level intersection. This paper analyses the basic requirements for road corridor design in the urban area, resulting in modern transportation solution.

Keywords: road corridor, traffic corridor, road design, traffic modelling, traffic planning, evaluation

1 Introduction

Primary road corridors in the cities are the main ones to accept and distribute traffic on which the entire transport system is based. Planning of a primary road corridor is time consuming process whose origins date back to the time of creation of urban areas and making of first urban plans. Over time, in the area of primary road corridor occur changes that cause the need for further evaluation, and traffic and construction.

Investment in transport infrastructure of cities in Croatia very often do not follow demographic expansion which results in traffic problems, considering that traffic volumes exceed the capacity of the system.

Planning of a primary road corridors as well as constant monitoring of events in the space is a basic prerequisite for the protection, preservation and quality of space, and ensuring sustainable urban development and the quality of the built environment. The primary road corridor in Zagreb is formed from the transport network between junctions Jankomir and Ivanja Reka or road Ljubljanska – Zagrebačka – Slavonska Avenue, which was once part of the Highway "Bratstvo i jedinstvo" and is one of the most burdened roads of Zagreb.

This paper describes the problems in planning of primary road corridors on the example of the City of Zagreb through the methodological approach and the selection of the optimal solution based on the analysis of space and integrated transport system. Valuation of transport networks leads to modern transport solution for the City of Zagreb.

2 Methodological approach

In planning of the transport system of cities, of which one component is the primary road corridor, decision must be based on a modern integrated transport model. Modern transport model allows simulation of traffic in the transport system for the base year, as well as forecasting the future state of traffic in the years in which particular scenario is tested. Based on the calculated traffic impacts of scenarios (variants) of the transport system development, the evaluation and decision on the optimal solution is performed. Methodological evaluation procedure is performed through the basic steps as follows:

· analysis of spatial planning and traffic-technical documentation;
· development of transport models;
· development of technical solutions;
· defining scenarios of the road network;
· analysis and evaluation of scenarios.

3 Display of choice of the optimal solution of the primary road corridor of the City of Zagreb

3.1 Spatial planning documentation

The basic spatial planning documents used in creation of the optimal solution and evaluation of the primary road corridor Ljubljanska – Zagrebačka – Slavonska Avenue (the Avenue) is Master Plan of the City of Zagreb (further in the text MP).
In the MP from 1970 (Figure 1) the corridor is categorized as "urban highway". The "urban highway" according to the MP is "expressly linking the eastern and western parts of the city with the city centre and connecting the eastern and western terrestrial highways" and "acceptance and distribution of traffic on the rest of the road network of the city via the connection of major roads that must be of high capacity".

Figure 1 Master Plan of the City of Zagreb 1970 – Basic road network

General construction characteristics of urban highway defined by MP from 1970 were: design speed of 80-90 km/h; lane width 3.50 m; principal corridor width 70 meters on both sides of the axis; dual carriageway with central belt width 4.00 m; the number of transit lanes depen-

ding on the forecasted load, at least three in each direction; all intersections are split-level intersections; distance of intersections minimum 1,000 m (exceptionally 600 m); pedestrian traffic longitudinal and spatially separated from motor traffic, transverse split-levelled; public transportation, is only possible rapid bus, without disturbing the main lanes.
In the current MP corridor is defined as the avenue with the following characteristics: corridor width of at least 40 meters; lane width 3.25 m; "intersection can be split-level intersections if required by traffic needs, and allowed by space capabilities"; "it is necessary to predict the rows of trees."
Analysis of the spatial planning documents, and comparison of historical and present data, shows, except reduction of characteristics of the primary road corridor, a noticeable and significant reduction in the width of the corridor (from 140 m to at least 40 m). Current corridor width of the Avenue according to the latest MP is shown in Figure 2.
As a result of the analysis of the corridor it has been established that east of Heinzelova corridor width is greater than 110 m. Looking westward from Heinzelova corridor width varies between 60 and 80 meters. The Avenue corridor is narrowest between Savska and Šarengradska with width of only 50 m.
The above described analysis shows that there is a lot of pressure on the idea of the primary road corridor in the cities, which results in the "confiscation" of space intended for transportation infrastructure. Ultimately, this result in additional costs and time required for implementation of the optimal solution, given that it is already spatially limited urban environment.

Figure 2 Zagrebačka and Slavonska Avenue corridor (up: Nehajska-Savska; down: Heinzelova-Čavićeva)

3.2 Description of present state

The Avenue was built as dual carriageway road, two lanes per carriageway, separated by a central reserve. Zagrebačka Avenue was reconstructed in year 2006 with upgrade of third lane on the stretch from Savska Opatovina to Selska.
On the whole stretch, length 22 km, 25 intersection was built (approximately every 900 m) of which 15 as signalized intersections in level. Split-level intersections were built mainly at the

intersection with avenues, with the exception of the intersection Ljubljanska Av. – extended Medpotoki, Zagrebačka Av. – Savska Opatovina and Slavonska Av. – Ljudevita Posavskog. Intersections were constructed over the last 30 years and in various forms (Figure 3): motorway loop "clover" with three levels (Držićeva), "trumpet" (Škorpikova and extended Medpotoki), the classic "diamond" (Savska Opatovina, Selska, Savska, HBZ and Posavska), "half-diamond" (Gospićka), "diamond" with the left detachment for the left turn (Heinzelova).
In the analysis of the classic "diamond" intersections design solutions are also not unified: classic four-way intersection (Selska, Savska, Posavskog), expanded four-way intersection (HBZ) and roundabout (Savska Opatovina). Such "mixed" design solutions of intersections as by type, as in the traffic management also represents a big problem for the driver and is one of the factors that influence the formation of traffic jams and reduce the safety of the existing transport system of described corridor.

Figure 3 Split-level intersections on Ljubljanska – Zagrebačka – Slavonska Av

3.3 Description of the planned project

Solution of the primary road corridor of the City of Zagreb is based on the reconstruction of Slavonska, Zagrebačka and Ljubljanska Avenue by reconstruction of the existing signalized intersections into split-level intersections and upgrading the number of required lanes on the main transit carriageways. In the area of coverage, from the Puljska in the west to the street Marijana Čavića in the east, the Avenue makes 16 intersections, 10 of which are split-level intersections and 6 connections, with the existing and planned road network.
The planned intervention in the initial step predicted two basic variants of reconstruction of the Avenue: "do minimum" (Variant 1) and "do maximum" (Variant 2). Based on the traffic analysis and spatial constraints of both variants, traffic segments that define the optimal solution (Variant 3) were selected.

3.3.1 Description of Variant 1 – "do minimum"

The main characteristic of Variant 1 is the upgrade of main transit carriageways to at least three lanes per direction and reconstruction of the existing signalized intersections into split-level intersections, Črnomerec and Kruge, with three lanes in each direction on main carriageways, construction of a new split-level intersection Šarengradska and construction of underpasses Sveučilišna – Miramarska – HBZ with 2 lanes per direction on the main carriageways. Existing split-level intersections are retained with two lanes per direction on main carriageways (Selska, Savska, Držićeva and Heinzelova).

3.3.2 Description of Variant 2 – "do maximum"

Variant 2, except upgrade of main transit carriageways to at least three lanes per direction, predicts reconstruction of existing signalised intersections into split-level intersections (Črnomerec and Kruge), reconstruction of the existing split-level intersections (Selska, Držićeva and Heinzelova), construction of new split-level intersections (Šarengradska, Sveučilišna and Miramarska) and construction of underpass Sveučilišna – Miramarska – HBZ with three lanes per direction on the main carriageways. In addition to the existing underpass Savska includes construction of a new underpass with three lanes for the northern carriageway of Slavonska Avenue.
Beside the upgrade of the third lane, Variant 2 predicts reconstruction of the intersection Držićeva from the motorway intersection in the classic diamond intersection typical for urban areas as shown in Figure 4.

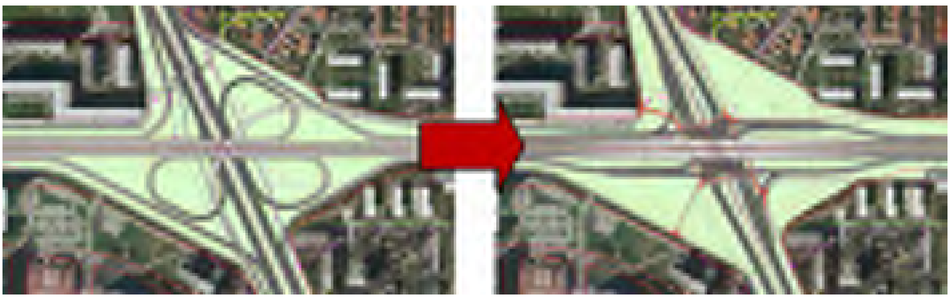

Figure 4 Reconstruction of intersection Držićeva

3.4 Selection of the optimal solution

In the area of research deterministic static simulation of individual traffic flows was made, where a wider area of the model covers an area from Vukomerečka road in the east to Zagrebačka road in the west. The wider scope of the model enables to test the traffic corridor or individual actions (scenarios) of the road corridor Zagrebačka and Slavonska Avenue and interventions in the immediate environment. This approach allows combining scenarios and finding an optimal solution in the metropolitan area.

3.4.1 Traffic analysis of existing corridor

Traffic analysis of the existing situation proves that the congestion on the Avenue in research area is mostly affected by signalized intersections Marohnićeva, Kruge and Čavićeva whose capacity is lower than other split-level network segments. In the segment of Slavonska Avenue between Držićeva and Heinzlova congestion is caused by the inability to distribute traffic to the north due to delays in Heinzlova Street towards intersection with the Vukovarsta street. Congestion can occur on ramps on Slavonska Avenue (Figure 5).

The planned increase of traffic volumes by 25% in year 2033 compared to the year 2013, with increased capacity of the Avenue, must be distributed through existing and planned split-level intersections.

Figure 5 Saturation of the network in year 2033 (Scenario 2033_V1-0)

3.4.2 Scenario analysis

Traffic analysis of future transport system predicted 8 scenarios. Basic characteristics of the scenarios are inclusion of both variants of the design solution for primary road corridor Slavonska and Zagrebačka Avenue (Variant 1 and Variant 2) in conjunction with actions in the network directly along the Avenue. This analysis leads to the conclusion that the functioning of the Avenue in the area of research to year 2033 is possible with reconstruction of existing signalized intersections into split-level intersections and construction of 2x3 lanes on the main carriageways, which would balance the traffic offer on transit east-west corridor.

Figure 6 Saturation of proposed network in year 2033

To solve the problem of distribution it is necessary to activate the planned network connected the Avenue; Šarengradska road from the Jadranski bridge to Vukovarska Avenue, Street Prisavlje from Marohnićeva to Marina Držića Avenue, Sveučilišna Street from Vukovarska Avenue to Street Prisavlje, Miramarska Street from Vukovarska Avenue to Slavonska Avenue, Street Kruge from Prisavlje Street to Slavonska Avenue. In the eastern part of the city must be activated the eastern part of the planned Zagreb ring road or section Vatikanska – Koledovčina. Saturation of the network in year 2033 for optimal solution of the Avenue and planned network is shown on Figure 6.

Table 1 shows the traffic effects for optimal solution of the primary traffic corridor in the City of Zagreb which predicts the reconstruction of the Avenue and activation of the planned network related to the Avenue. It turns out that the savings in the transport system in the year 2013 are 220 million € / year. With the increase of traffic in year 2033 effects will be 292 million € / year. The above indicators are evidence of the need for investment in primary road corridor in the City of Zagreb and the development of the network in the immediate vicinity.

Table 1 Traffic effects of the primary traffic corridor in the City of Zagreb

	[illegible]				[illegible]			
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]								
[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	
Total	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	
[illegible]								
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Total	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[illegible]

3.5 Description of the optimal solution of the primary road corridor in the City of Zagreb

The optimal solution of the primary road corridor Zagrebačka and Slavonska Avenue represents Variant 3 (Figure 7) which predicts reconstruction of signalized intersections Črnomerec and Kruge into split-level intersection with three lanes per direction on the main transit carriageway, construction of underpasses Sveučilišna – Miramarska – HBZ with 3 lanes per direction on the main carriageway and reconstruction of existing split-level intersection Heinzelova in classic diamond intersection typical for urban areas.

Figure 7 Slavonska Avenue

Geometrical characteristics of the corridor are based on the design speed; 80 km/h on the main transit carriageways and 50 km/h for the service roadways and intersection ramps. The main carriageway of Zagrebačka and Slavonska Avenue contains a minimum of three lanes per direction. On the ramps of split-level intersections number of lanes depends on the forecasted traffic volume, a minimum of two lanes. The cost of construction of Variant 3 is estimated at an amount of 73,4 million €. The project can be realized in three phase. Review of priorities and their costs is given in Table 2.

Table 2 Review of priorities and the cost of reconstruction of the Avenue

Phase	Section	Start	End	Length	Cost
I.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
II.	[illegible]	4+650	[illegible]	[illegible]	[illegible]
III.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	Total			[illegible]	73,40 mil.€

4 Conclusion

Integrated planning and management of spatial and transport factors allows creation of optimal solutions for the transport system of the city as a whole, which contributes to sustainable development and to improve the efficiency of spatial and transport planning. Revision of existing and planning of new road corridors in the transport system must be evaluated through a modern integrated transport model. The traffic model is the basic "tool" in planning procedure, under which it is possible to calculate the traffic impacts of the proposed idea. This paper describes a practical example showing that corridors planned in spatial plans require further evaluation, and traffic and technical. The result of evaluation often requires an amendment to the spatial planning documentation which is causing an increase in cost and time for implementation of the project.

References

[1] Ministarstvo zaštite okoliša, prostornog uređenja i graditeljstva, www.mzoup.hr

[2] Grad Zagreb, Zavod za prostorno uređenje Grada Zagreba: Prostorni plan Grada Zagreba – izmjene ii dopune, 2009.

[3] Grad Zagreb, Zavod za prostorno uređenje Grada Zagreba: Generalni urbanistički plan grada Zagreba, 2009.

[4] IPZ d.o.o.: Idejno rješenje Ulice Črnomerec od Jadranske avenije do Zagorske ulice, Zagreb 2013.

[5] AKING d.o.o., Sveučilište u Zagrebu Građevinski fakultet: Idejno rješenje rekonstrukcije Slavonske i Zagrebačke avenije od Ulice Marijana Čavića do Puljske ulice, Zagreb 2013.

[6] Njegovec, M.; Majstorović, I.; Stepan, Ž.: Prometno planiranje za utvrđivanje infrastrukturnih koridora, Dani prometnica 2011 – Projektiranje prometne infrastrukture, Zagreb 2011., str. 229-252

STRATEGY OF DEVELOPMENT TRENDS IN THE MODERN CITY – A GREEN TRANSPORT PLAN IN CASE OF ZAGREB

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Abstract

Strategy of development trends in this moment and at this stage of the traffic comes from understanding the possibilities of the implementation of measures to raise the quality of movement and life in the city, planning and implementation of targeted interventions in the transport network or facilities, as well as design of public spaces in the city that will significantly affect the improving the quality of movement and urban life. The prerequisite for the planning the urban transport is knowledge of a large number of parameters and factors, among which is the central factor which is mobility of the participants in traffic. Mobility is a complex process in the area, starting from the departure out of the apartment, through the use of various means of transportation, a number of activities on different goals until the return to the location of the apartment. It thus requires extensive transport infrastructure in the form of pedestrian, cycling, street and railway network. At the same time reshaping of public transport facilities is needed in which the pedestrian, biker and green shall be the main elements of the new spatial conditions. "Developing of the pedestrian zones in the city" is the concept of creation of pedestrian areas and islands in all the places in the city where it shall be possible. This process is essential for the transformation of public spaces, in particular transport corridors. To accomplish the concept of the ''green transport plan", the need for change in the parking policy is emphasized, which should be an integral part of the city planning and should go hand in hand with the policy that offers good alternatives to car use, reducing street parking along the main roads and adjustment of regional plans to implement the construction of parking spaces outside streets.

Keywords: strategy of development trends, green transport, pedestrian zones

1 Introduction

Today, moving through the city is manifestation of freedom of choice of each individual and is proportional to the number of possibilities for realization, desires, needs and aspirations in relation to the destination point. Two groups of parallel movement processes in the city are taking place that are seemingly opposite. The first movement is related to the rational selection of the path which is the shortest in time for coming from one point to another (functionality). The second group includes the often unplanned movements or remaining in the interiors of public spaces which reflects the realization of the aspirations of the city inhabitants for a comfortable and pleasant life in the city (emotion). The quality of movement and the presence in the city public areas should be explored and should reflect in the harmonization of these two groups of processes.
Urban morphological city sequences occurred throughout history have formed the streets, squares, parks, promenades, coasts, sports and recreation facilities, etc., therefore all public

or common city areas in which collective life of its inhabitants is carried out. These are the SPATIAL CONSTITUENT CITY ELEMENTS. Architectural forms, such as buildings of all kinds and types are CONSTITUENT ELEMENTS OF TYPOLOGY, [1, 2]. The content structure that encourages movement in this complex figure shapes urban processes of endless dynamics of people and goods. These intensive and dynamic processes require permanent care for quality improvement of public spaces. In this sense, the interdisciplinary approach basis is the basis for work on complex analysis of planning and design of traffic areas in the city [3].

2 Spatial traffic analysis of the city of Zagreb

Traffic Study of the Northern Tangent in 2006 and Traffic Study of Zagreb in 2008 [2, 3] and by the analysis of the territory, natural and artificial factors in the area were processed that were considered to have significant impact on the development of the city as a whole and thus traffic.
Territory in which the city beneath Zagreb's mountain develops, up to and across the river of Sava is over 40 km long in the east – west and wide by 3-5 km in the north – south direction. This elongated territory area of approximately 200 square kilometers intersects a number of significant longitudinal and transversal routes supplemented with several diagonal routes. Emphasized longitudinality of city development is a logical consequence of its geographical location, layout of the content, population and activities in the area.
In the proposed network, significance have the transport routes of two central longitudinal speed city roads Zagreb – Ljubljana – Slavonia Avenue and the planned extended Branimir Avenue along the railway line, which is the backbone of the city's transport system. In the northern perimeter of the city the northern tangent is planned, and a bypass highway was built in the southern border of the city. Longitudinal directions in the city net are cut by ten transverse directions and diagonals that typically connect the city areas by bridges on the left and right bank of the city.
Basic transport network is supplemented by a series of streets whose primary function is to provide movement of traffic, but also the formalization of the built environment in which the street network is an equal part of a complex assembly of public space in which daily life of citizens is carried out.

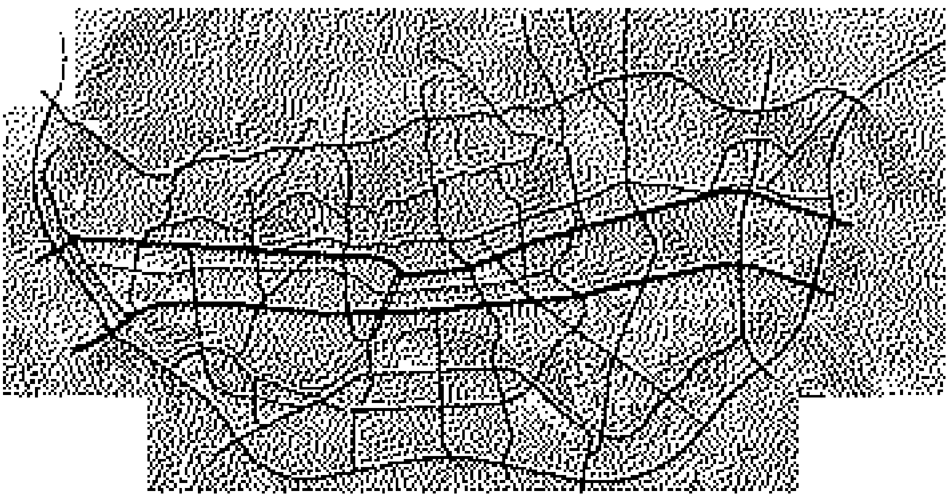

Figure 1 Schematic overview of the transport network (proposal) [4]

3 The necessity of planning the urban transport

The prerequisite for the planning of urban transport is knowledge of a large number of parameters and factors, among which the central is the mobility of citizens which requires extensive transportation infrastructure in the form of hiking, biking, road (street) or the rail networks, and parking areas. The construction and expansion of infrastructure leads to a new traffic offer and enables new ways of usage. Planning is a way of attempting to intervene in this complex process by introducing a systematized order with alternative conceptions. Immanent part of the traffic planning system is the conflict that arises from the diverse ways people react to the planned measures. On the one hand, planning must try to influence the behavior patterns of people, and on the other hand planning must adapt to the ways of behavior. Today we can say that despite all efforts we failed to adequately solve the problem of Zagreb and traffic and to achieve planned goals [5, 6, 7].

3.1 Conflicts between transport and public space

Conflicts between transport and public space appear in the planning goals of safety, efficiency and capacity of individual means of transportation, and in their relation to each other. Through the surface they need they directly change the road space. Width of lanes and their number, extension of nodes, separate paths for pedestrians and cyclists, overpasses and underpasses are in conflict with the available space of roads and squares. But the problem of traffic in the city should not be reduced only to the conflict over the formation of the limited street space. The roots to overcome the problems reach deep, all the way to the overall planning of urban transport and space.

The main street network in Zagreb's city center is planned and built in the 19^{th} and at the beginning of the 20^{th} century and by its traffic and technical characteristics meets today's traffic needs. Through all that time, there was no planning and construction of public parking, which today, because of the lack of parking spaces, stands out as a major traffic problem.

Surface area of the city center (the historic center) is about 2% of the total area of the city and covers about 10 % of all tertiary activities. Therefore, in the historic center it is not possible to suspend the traffic, but conduct a selection that would allow access to business entities and flats. Unfortunately, it is in this part that the planners of Zagreb failed even though they were on a good way to solve the practical problem of parking, starting from the DUP (General Urban Plan) Center Zagreb in 1974 and general transport plan in 1978. This plan planned to build a garage with total capacity of approximately 25,000 parking spaces. Subsequent amendments to the General Urban Plan departed from this concept, and the dotted construction of garages was allowed, which was later completely prohibited, while the expansion of pedestrian zones and creation of even bigger problems in the parking of tenants and businesses entities in the central area continued [8].

Today, in the central city area covered by the collection of parking, a total of 51,969 cars are registered. If you add to this approximately 34,000 cars coming to Zagreb daily there is a need for the subtler managing transport demand and its satisfaction.

Construction of garages in the city of Zagreb is particularly delicate issue because of the different approaches of people, professions and investors. Decisions of administrative bodies should be based on existing and new traffic analyzes and studies that provide the possibility of building the so-called block garages, which would partly be used by residents, and partly would be available for public parking. It is necessary to determine the necessary capacity of these garages and construction stages so that the first stage terminates up to 50% of street parking spaces. This would reduce the “visual pollution” of the city and allow the development of the concept of “Developing of the pedestrian zones in the city”, i.e. the creation of pedestrian zones and green islands in all the places in the city where possible.

3.2 Reshaping of public city spaces

The city streets are primarily corridors of unhindered movement of people, vehicles and goods, but also of daily events in the urban life of the city. Road space is more than traffic area, it is a place where people are present, and it is the area of communication and perceiving the image of the city. Public city spaces form the urban landscape and contribute to the creation of unique environments that are the key to the experience and memory of the city [9].
Intertwining in the town by green corridors, which is also part of the movement of people network, is a good basis for urban mobility as one of the most important urban processes in the identification of population with the city. By developing an idea about the city, concepts of green areas of the city develops as a network of eco-systems of different types of greens. Categorization of the planned network and detailed standards for equipping and forming the city streets are processed in the Traffic Study of the City of Zagreb in 2008.

4 Green Plan and turning the city of Zagreb in the pedestrian zone

Corridors of greenery are one of the most important elements of transformation of street outlines and public spaces. Greenery as a cityscape element significantly changes the physiognomy of the city scenery area from which parked cars disappear disposed within the newly built garage parking space. Green alleys in the historic parts of the city or linear outlines of freely formed groups of greenery in its newer parts are the welfare of the city and its ambientalization, [10, 11].
During the historical periods of Zagreb development extraordinary patterns and greening typology of the series of streets, squares, etc. were created. The need to continue designing public spaces by greenery in contemporary urban development concepts of Zagreb has already been featured in directive regulatory basis in 1949/1953 and in the basic documents of the urban development of the city that followed thereafter, [12]. The present time requires significant reshaping of street outlines and other public spaces by greenery which should be weighed following the application of propositions from the categorization proposals of the transport network for the city and the concept of greening and developing the pedestrian zones of the city.

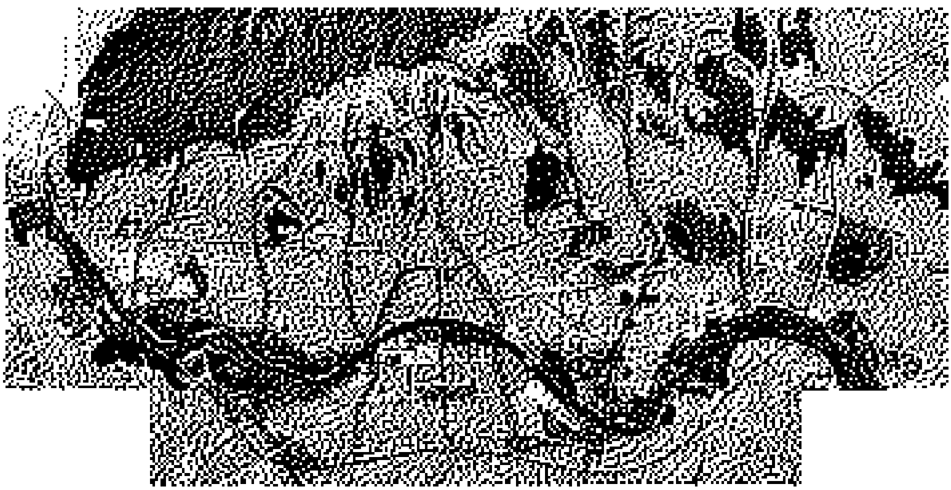

Figure 2 Green Areas of the city and transverse transport corridors [11]

This approach of solving the problem of movement in the modern city a proposal for a new concept of decorating public spaces is created which can be called "GREEN PLAN AND DEVELOPING OF THE PEDESTRIAN ZONES OF THE CITY". There are a number of zones and street outlines that become pedestrian oasis by simple traffic reorganization.
These pedestrian oases equipped by urban mobiliars, richly planted and connected by walkways and bicycle paths contribute to the quality of life of residents in the city with the additional capability of movement and presence in the city as part of a comprehensive process of the residents intramobility. In this sense, Zagreb has a huge number of still unused opportunities shown by the most basic analysis of the possibilities of forming a system of pedestrian area of the city. Developing the process of making the pedestrian zones of the city is possible to begin at approximately 30 locations that are deployed throughout the metropolitan area on the spacing of 500 m to 1000 m. Along with pedestrian movement, forming of bike route networks is especially suitable in these distances which should connect these exceptional places of leisure time, aesthetic appeal, sociability, identity and environmental awareness of residents of the urban neighborhoods. In this regard, the case of the city of Vienna should be noted which, through the process of urban renewal in the last 15 years, realized more than 40 such protected pedestrian zones and islands, [13].

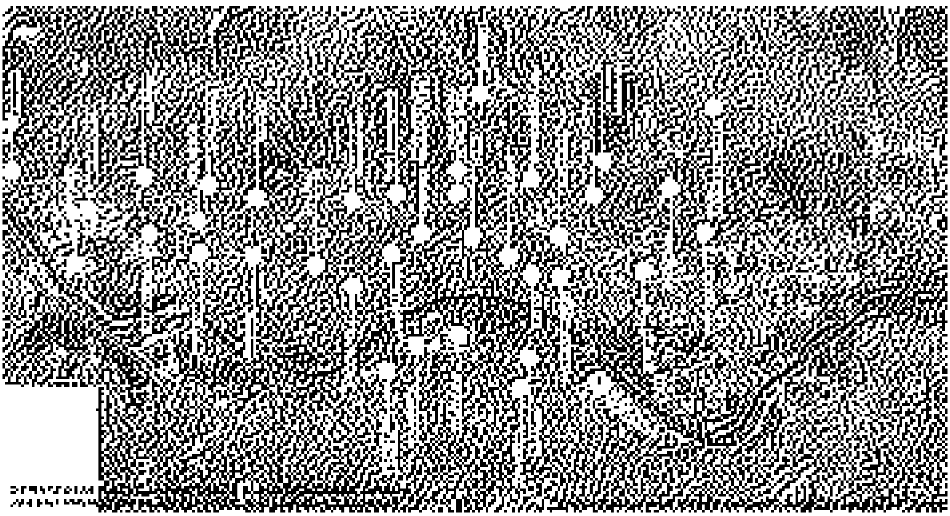

Figure 3 Schematic overview of the network of possible locations of pedestrian areas

5 Transparency in decision-making about the regulation of the transport area

Implementation of the proposed green transport plan and developing the pedestrian zones of the city as a socio-cultural intervention of largest urban values shall be, by experience, difficult to implement. It is therefore necessary by continuous dialogue through the media, organized forums and specific forms of communication, such as social entrepreneurship as a new paradigm in the approach of solving social problems, engage as many residents of the city as the end users of the proposed changes [14, 15]. In particular, it relates to households, companies and institutions that are feeling the effects of the planned measures. In this way it is possible to form relationships between holders of traffic planning affairs, project contractors and environment in accordance with the objectives of the planned activities, and consequently stimulate the preferred way of environmental conduct by which the conflicts are reduced to the lowest possible level, [16].

6 Conclusion

The essence of the proposal of the green transport plan and developing pedestrian zones of the city is focused on the creation of the movement organization (traffic) while decorating the town in which the factors of economic, social and environmental development shall act on spatial shaping of the idea about the city. At this time, significant and highly valuable urban areas are occupied by parked vehicles. It is believed that the removal of a large number of vehicles from the outstanding parking would be replaced by parking in multi-story parking garage facilities which would release a very large surface area that can transform itself into a pedestrian zone and urban greenery and form a new urban quality and valuable ecological category by equipment. In this way, parking of vehicles would be controlled in a far greater metropolitan area than just the city center. The network of pedestrian areas and greenery of street corridors linked with city parks, forest parks, river banks, meadows and other types of green areas would ensure the city a higher quality of life for its residents.

Literature

[1] Rossi, A.: The architecture of the city, Biblioteka psefizma, Zagreb 1999

[2] The center of Zagreb, Detailed urban plan, Head architect B. Kincl, UZGZ Zagreb 1976

[3] Elvin, G.: Integrated practice in Architecture, Inc. 2007, Hoboken, New Jersey

[4] Spatial – traffic study of road and railway system of the wider area of Zagreb, IGH and AF, Department of Architecture,''Town and Country Planning'' Head academic B. Kincl, Zagreb 2008

[5] Steierwald, G., Kunne, H.D.: Stadtverkehrs-planung, Teil A,Springer-Verlag, Berlin Heidelberg, 1994.

[6] Vanderbilt, T.: Driving. Algoritam, Zagreb, 2011

[7] General traffic plan of the city of Zagreb, 1978

[8] Matoš, S., Šimurina, S.: Development of the system of parking charging in the City of Zagreb by changing the use of privileged parking tickets, Proceedings, Automation in Transportation, 2012, Zagreb-Vienna

[9] Cerasi, M.: Lo spazio collettivo della citta, Gabriele Mazzotta editore, Milano, 1976

[10] Kincl, B.: The Art of Breathing and design of the city in Zagreb example, International Scientific Conference ''Greenery of Zagreb", HAZU, Zagreb 2013

[11] Jacobs, B.A.: Great streets, MIT, Cambridge, Massachusetts 1993

[12] Ivanković, V.: Concept of green Zagreb in the vision of the architect Vladimir Antolić, International Scientific Conference ''Greenery of Zagreb", HAZU, Zagreb 2013

[13] Urban – space – experience / Design for public space in Vienna / Vienna urban development and planning, Vienna 2003

[14] Bežovan, G.: Civil society, publishing house Globus, Zagreb 2005

[15] Dakić, S.: Strategic issues and challenges of the development of Zagreb, Conference Development of Zagreb, 2008

[16] Matoš, S., Tomšić, D.: City Traffic Planning and Public Relations, Proceedings, The Fifth Congress on Croatian Roads, Cavtat-Dubrovnik, 2011

GENETIC ALGORITHMS TO OPTIMAL DEFINITION OF PEDESTRIAN TERMINAL LAYOUT

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Abstract

In the recent years is observed a growing interest in the analysis of pedestrian dynamics. Nowadays architecture design must aim to identify constraints and requirements to satisfy pedestrian flows inside nodal infrastructure – such as train stations, subways, airports – as well as into any civil complex that may gather notable pedestrian flows – city malls, theatres, meeting rooms, skyscrapers – and in urban elements such as shopping streets, main important squares, pedestrian flows in case of exceptional events, etc. In order to create an instrument to support technicians into pedestrian terminal projects and optimum layout definition according transport needs, an interesting approach was developed. Starting from analytical background for facilities dimensional range definition, through pedestrian dynamics modelling and cost objective function definition the application of genetic algorithm combined with social force models allows to identify a domain of optimum solutions compatible with pedestrian needs.

Keywords: pedestrians, terminals, design, optimization, genetic algorithm

1 Introduction

Nowadays, it is given major attention to pedestrian dynamics. Day by day, millions of pedestrians move in the world with different purposes and behaviors. Various disciplines and research activities are dealing with these items. Pedestrian behavior, reason of their movement, path selection and the necessity to model them, crowded phenomena, emergency situations etc, are very active sectors in transport engineering, in psychology, in economy as well as in computer vision techniques, biology etc. The complexity of pedestrian dynamics increases according to the intensification of the flow and, consequently, of the density. Interaction among pedestrians and their reaction to the objects present in the environment where they move are the motivation of numerous theories. The results are not only statistical data and qualitative considerations, but also models both analytical and simulative. Pedestrian dynamics conveys in area where interaction can increase and decrease rapidly as for example in urban environment, – squares, pedestrian zone, commercial streets – pedestrian infrastructure as well as complex and large buildings – skyscrapers, directional and recreational buildings, cinemas, theatres, city malls – stadium etc. Engineering, architecture and specialized techniques work every day very closely to project the works above mentioned. Terminal, such as airport, subway station, railway station, bus station, cruise terminal etc, is a place where lots of pedestrians move inside for different purposes and in different ways, where interaction are relevant and where crowded phenomena and emergency possibility are ordinary items. While the goals of functionality and flexibility have a primary importance, the planner must also consider ways of creating a building layout and environment that supports the highest levels of passenger services and facilities in balance with the size of the building envelope and available budget. After a deepen and careful analysis of literature appears clearly that there is a great lack about the concept of optimum terminal layout definition, while deepen study were carried out to analyze single facilities like escalators, elevators, walkways, doors performance, stairs etc.

2 Methodology developed

The methodology developed allows to the user to arrive at the definition of the best layout configuration according input desired in some consecutive steps. First of all, it must be selected the case where the process must to be applied: e.g. if the objective is the identification of the best terminal layout for an underground terminal, the user will define a basis layout of the same terminal with the boundaries and the identification of the point where it will be inserted characteristic elements. In a pedestrian terminal will be defined as characteristic elements all the elements that generate consequences into pedestrian flow and dynamics as for example platforms, stairs, escalators, elevators, walkways, doors etc. Secondly, it must be defined the Level of Service – L.O.S. – desired that can be a single value or a range. This must be defined for every single characteristic element according to H.C.M. – Highway Capacity Manual – approach. In order to identify the volume of pedestrian that will move during the life useful of the terminal is required the quantification of the flow for dynamics that will be generated inside. Selected the above inputs, through an analytical model based into H.C.M. and T.C.Q.S.M. – Transit Capacity and Quality Service Manual – the dimensional range of every single characteristic elements is defined. The further application of an analytical model defines the travel time for pedestrian in usual condition, comfort condition as well as into emergency situation, increasing the flexibility of the approach. To evaluate the behavior of the terminal, according previous inputs, in terms of infrastructure costs and pedestrian costs, an objective function was elaborated. This aim to correlate the variation of terminal sizing with pedestrian travel time. Consequently, SOBOL methodology was used to generate the population for the next genetic algorithm application. This is possible through the application of the analytical models above described that provide results for objective function. Starting from SOBOL results, the research of the objective, best terminal layout configuration through the minimization of objective function, is pursued with M.O.G.A. – Multi Objective Genetic Algorithm. This led to identify a Pareto front with optimum solutions domain. At this point, the necessity to determinate the final performances of the terminal and consequently select the best solution led to apply the social force algorithm. This allows to obtain qualitative and quantitative measurements that led to final selection.

3 Analytical Tool

The issues concerning pedestrian flows have begun to have some relevance only in the last 40-50 years. In a first approximation, the studies concerning the analysis of the characteristics of pedestrian traffic can be divided into two broad categories (May, 1990): macroscopic and microscopic analysis. The first studies were based on direct observation and on the use of photographic material or shooting with cameras in slow motion. These studies gave rise to the first macroscopic models: they consisted in the development of the concept of “Level of Service” (Fruin, 1971), in deducing design rules for optimal pedestrian infrastructure, or in the development of guidelines for planning and design of transport systems (HCM, 1985). Moreover, these models took into account the traffic flows in the aggregate, without taking into account the inevitable interactions that are established between individual pedestrians, and that influence their behavior. Therefore, a further step forward was the introduction of so-called microscopic models: mathematical models of simulation of pedestrian behavior, both in normal and emergency conditions. One among the first and most important of these models assumed for the movement of pedestrians a behavior similar to that of the gases or fluids (Henderson, 1974): this model was born first as macroscopic model, then adapted to take into account the interactions between the different entities (Helbing, 1992). Later were further developed mathematical models, such as models of the "Code" (Yuhaski, Macgregor Smith, 1989), models in the transition matrix (Garbrecht, 1973), stochastic models (Ashford, O'Leary, McGinity, 1976). In recent years have emerged models that currently constitute the most used tools in the analysis in the field of pedestrian models: model with cellular automata (Blue and Adler, 2000; Muramatsu et al., 1999) and the model of social forces (Helbing, 1991) which, together with models based on artificial intelligence and research models of the path (way-finding algorithms), form the basis of modern simulation tools of pedestrian traffic. A terminal can be defined as a sequence of characteristic elements through which pedestrian flow move. A good equilibrium in terms of capacity/potentiality of these elements imply a good or bad performance of the terminal in terms of L.O.S. subjected to a data demand. Starting from these general considerations, the need to identify dimensional range for facilities inside which test pedestrian behavior in relationship to their paths and interaction, pursued to develop an analytical tool easy to be used based into main confirmed approach as H.C.M and T.C.Q.S.M.

Figure 1 Facilities dimensional range definition

Defined a L.O.S. – Level of Service – desired, identified the demand and particular environmental conditions, as for a platform can be surface not available for pedestrian movement, space reserved for disabled etc. the methodology allows to the user to obtain a dimensional range for every single characteristic element of the terminal. This domain of possible acceptable solutions, will be the starting point for the best solution identification. In order to understand the behavior of pedestrian trough the terminal, an analytical model was used to get users travel time inside infrastructure. This imply the possibility to evaluate as every single facilities satisfy the flow and how the flow move through them. Incorrect dimension led the pedestrian to have delay into facilities passage increasing the total travel time from origin to destination. This objective was carry out with the approach developed from V.M. Predtechenskii A.I. Milinskii [1] and in a next moment reviewed and applied from DDr. U. Schneider [2] for evacuation dynamic. Even if this model principally is used to evaluate evacuation process, through speed conversion is possible to consider dynamics also under comfort and usual conditions.

Figure 2 Pedestrian dynamics modellization through analytical model

4 Genetic Algorithm

Defined dimensional range of characteristic elements and identify pedestrian travel time inside terminals, an objective function finalized to minimize the sum of terminal construction cost and pedestrian travel time cost was developed. This function allows to apply the genetic algorithm. Genetic algorithms belong to the larger class of evolutionary algorithms (EA). Evolutionary algorithms (EAs) are population-based meta heuristic optimization algorithms that use biology-inspired mechanisms like mutation, crossover, natural selection, and survival of the fittest in order to refine a set of solution candidates iteratively. [3-6] Genetic algorithms (GAs) generate solutions to optimization problems using techniques inspired by natural evolution, such as inheritance, mutation, selection, and crossover. GA are a subclass of evolutionary algorithms where the elements of the search space G are binary strings (G = B*) or arrays of other elementary types. The genotypes are used in the reproduction operations whereas the values of the objective functions f € F are computed on basis of the phenotypes in the problem space X which are obtained via the genotype-phenotype mapping. [7,8,9]. The evolution usually starts from a population of randomly generated individuals, and is an iterative process, with the population in each iteration called a generation. In each generation, the fitness of every individual in the population is evaluated; the fitness is usually the value of the objective function in the optimization problem being solved. The more fit individuals are stochastically selected from the current population, and each individual's genome is modified (recombined and possibly randomly mutated) to form a new generation. The new generation of candidate solutions is then used in the next iteration of the algorithm. Commonly, the algorithm terminates when either a maximum number of generations has been produced, or a satisfactory fitness level has been reached for the population.

5 Application and results

Let to consider a portion of an underground station. During a traditional path, pedestrians move from trains to exit and vice versa. Defined the layout boundary and the characteristic elements through which flow moves, as for example into a path from train to exit could be platform, walkways, escalators, turnstiles etc, with H.C.M. and T.C.Q.S.M. approach above described the dimensional range of every single characteristic element is obtained, in respect of the flow and L.O.S. required. Being the characteristic element sizing allowed inside the range above identified, several combination of terminal layout can be generated. For every one of these the travel time for pedestrian can be evaluated as well as terminal performances. The application of Sobol Design of Experiments generates different layout configurations, each of them with a specific dimension of characteristic elements. This imply that for every single configuration a surface is determinated and the consequence travel time for pedestrian is obtained. Sobol methodology allows to generate population – possible terminal layout configurations – in order to apply M.O.G.A. Algorithm, Multi Objective Genetic Algorithm, for optimization process, according objective function minimization. The M.O.G.A. is applied to the objective function that considers the infrastructure cost, function of infrastructure surface, generated from combination of characteristic elements, and pedestrian cost, function of travel time. The minimization of objective function allows to obtain a Pareto front with the optimum solution identification.
In most real-life optimization situations, however, the cost function is multidimensional. Consequently, there is no unique optimal solution but rather a set of efficient solutions, also known as Pareto solutions, characterized by the fact that their cost cannot be improved in one dimension without being worsened in another. The set of all Pareto solutions, the Pareto front, represents the problem trade-offs, and being able to sample this set in a representative manner is a very useful aid in decision making, [10].
In the following picture the result obtained for a case of study done are plotted. Required a level of service D-E, Sobol and consequently M.O.G.A. allows to obtain different terminal configurations that minimize objective function. The point in the following graphic are the possible terminal layout configurations, the green point represent the Pareto Front, optimal solutions.

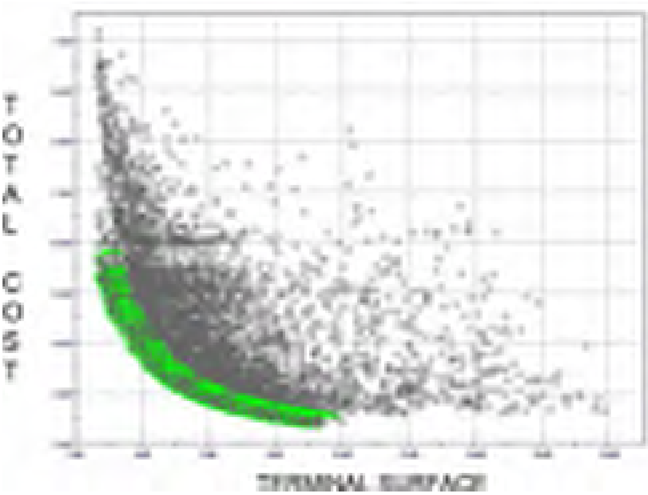

Figure 3 Pareto front obtained with strict range values for input variables

Even if all the solutions, taking into account the way in which dimensional range for D.O.E. were set up, are compatible with LOS that the terminal should have considering its pedestrian flow, is clear that the decision of the best solution cannot be taken only according cost values or pedestrian travel time.
Identified the optimum solutions, an analysis about the performance of the terminal was carry out with Social Force Algortihm. Run the simulations for every optimal scenario a lot of information can be obtain to evaluate the terminal performances.
The analysis, helpful to evaluate alternatives, are qualitative analysis as paths and trajectories, and quantitative analysis as travel time, average speed, speed los, distance walked from every single pedestrian, average minimum and maximum density and counters.
Divided the layout into potential cell of equal size is obtained the number of cell with the same LOS, during simulation windows time. The surface obtained is a good indication about the general performances of the layout that helps technicians to select the best layout configuration.

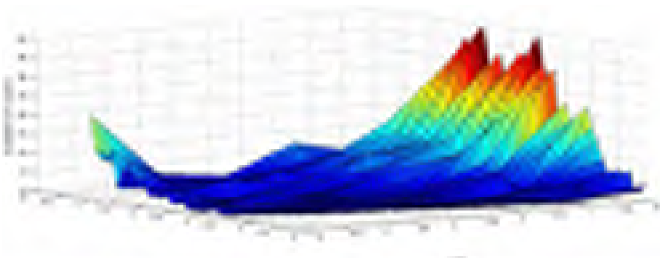

Figure 4 LOS representation for a possible scenario

In the following figures some outputs are shown. Into the fugures on the right the reader can see the results for a layout generated from the experience, while into the figures on the left are plotted the same result for an optimal solution that allow to minimize surface assuring an acceptable travel time for pedestrian according the required L.O.S. and flow.

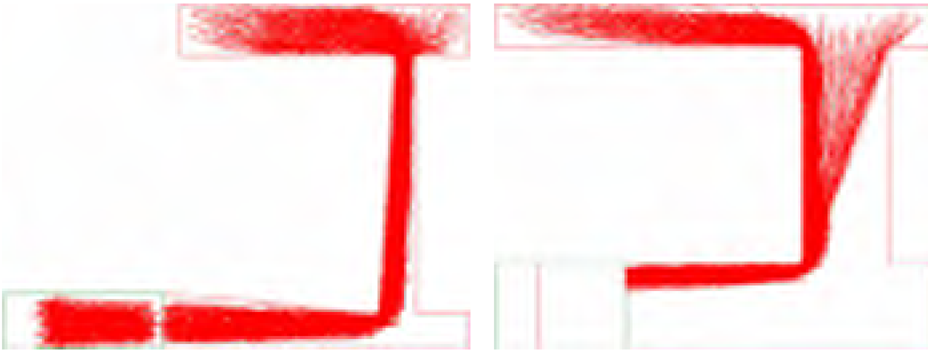

Figure 5 Trajectories into optimal scenario – left – and standard scenario – right

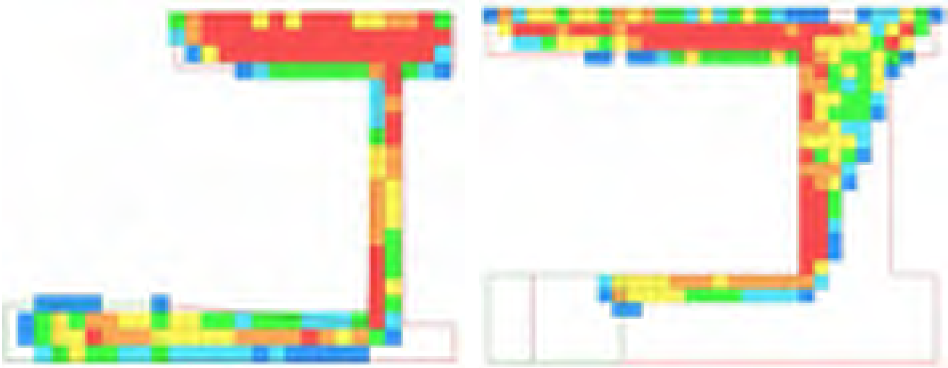

Figure 6 Max density into optimal scenario – left – and standard scenario – right

6 Conclusion

The major attention for pedestrian dynamics requires more reliable and useful techniques to find solution at daily problems. Congestion situations, delays, spaces undersized etc. are frequent in different scenarios. Starting from H.C.M. and T.C.Q.S.M. approaches, it was defined a tool able to identify the dimensional range of characteristic elements inside terminal layout according to Level of Service required and pedestrian flow prevision. Represented the pedestrian dynamics through an analytical model that allows to define the travel time for pedestrian in usual conditions as well as into emergency and comfort situations, it was defined an objective function finalized to identify the right trade off between infrastructure and pedestrian costs. The behavior of the function was tested through Sobol approach, necessary to generate population for next genetic algorithm M.O.G.A. application.
The domain of possible optimal solutions was evaluated through social force algorithm, that allows to obtain qualitative and quantitative indicators for an objective terminal performances evaluation. The methodology developed allows to approach terminal project definition, its analysis and renovation process, from another point of view, thanks to the application of genetic algorithm applied into the objective function defined on analytical basis. The process led to obtain optimal solutions about layout configuration according Level of Service desired and flow prevision, sometimes also unexpected and difficult to be predicted. Every solution rising from analytical background through the Sobol and M.O.G.A application is tested and evaluated with Social Force algorithm simulative approach. The outputs obtained are very

useful to evaluate terminal performances in an objective way, through numerical indicators based into H.C.M. and T.C.Q.S.M. approach. The test done in the case of study examined shows that the methodology is able to provide sizing requirements to be adopted in terminal layout configuration, finalized into the satisfaction of the demand required, researching the right trade off among the magnitudes involved into phenomena.

References

[1] Predtechenskii, V.M., Milinskii, A.I.: Planning for foot traffic flow in buildings, New Delhi: Amerind, c1978.

[2] Schneider, U.: Evakuierung bei Brandereignissen, Institut fur Baustofflehre, Bauphysik und Brandschutz TU Wien, Vortag TAE, March 2004

[3] Back, T.: Evolutionary Algorithms in Theory and Practice: Evolution Strategies, Evolutionary Programming, Genetic Algorithms. Oxford University Press US, January 1996. ISBN: 0-1950-9971-0, 978-0 19509-971-3.

[4] Back, T., Fogel, D.B., Michalewicz, Z.: Handbook of Evolutionary Computation. Computational Intelligence Library. Oxford University Press in cooperation with the Institute of Physics Publishing / CRC Press, Bristol, New York, ringbound edition, April 1997. ISBN: 0-7503-0392-1, 978-0-75030-392-7,0-7503-0895-8, 978-0-75030-895-3.

[5] Back, T., Hammel, U., Schwefel, H.P.: Evolutionary computation: comments on the history and current state. IEEE Transactions on Evolutionary Computation, 1997; 1(1): pp. 3–17.

[6] Mitchell, M.: An introduction to genetic algorithms. Cambridge, MA: MIT Press. ISBN 9780585030944. 1996.

[7] Holland, J.H.: Adaptation in natural and artificial systems: an introductory analysis with applications to biology, control, and artificial intelligence. The University of Michigan Press, Ann Arbor, 1975. ISBN: 0-4720-8460-7, 978-0-47208-460-9, 0-5850-3844-9, 978-0-58503-844-5, 978-0-26258-111-6. Reprinted by MIT Press, April 1992, NetLibrary, Inc.

[8] Heitkotter, J., Beasley, D.: Hitch-Hiker's guide to evolutionary computation: a list of frequently asked questions (FAQ). ENCORE (The EvolutioNary Computation REpository Network), 1998. USENET: comp.ai.genetic.

[9] Whitley, D.L.: A genetic algorithm tutorial. Statistics and Computing, 1994; 4(2): 65–85. ISSN: ISSN 0960-3174 (Print) 1573-1375 (On-line). doi:10.1007/BF00175354.

[10] Legriel, J., Le Guernic, C., Cotton, S., Maler, O.: Approximating the Pareto front of multi-criteria optimization problems? Tools and Algorithms for the Construction and Analysis of Systems Lecture Notes in Computer Science Volume 6015, 2010, pp. 69-83

ASSESSMENT OF THE DEMAND FOR BICYCLE PARKING INFRASTRUCTURE IN VIENNA

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Abstract

One of the official goals of the Viennese transport policy is to increase the share of cycling by more than twofold. Investments into cycling infrastructure are the key to success. Besides cycling paths and lanes the necessary infrastructure also includes safe and secure parking facilities. Appropriate bicycle parking facilities are needed at primary locations (home) as well as secondary locations (work, shopping, leisure, etc.). The Research Center for Transport Planning and Traffic Engineering, Vienna University of Technology, recently carried out two different studies concerning the assessment of the demand for bicycle infrastructure. The aim of the proposed paper is to present the results of these two studies.
The starting point is an analysis of the legal framework for on- and off-street bicycle parking in Austria. Existing planning guidelines are compared with international examples from countries and cities with very high shares of cycling. Citywide data about the location of public bicycle stands are analysed. Six case study areas in the city centre, the inner city area, the suburbs and at a main railway station have been defined. Occupation rates of the bicycle stands in these areas have been counted and analysed. A web based survey has been carried out in order to gain data about bicycle parking at private locations (home, workplace). The spatial distribution of the future levels of cycling has been estimated using three different methods. According to the results of our research a total of about 44,000 to 56,000 additional public bicycle stands are needed to accommodate the intended increase in cycling. The highest demand has been identified for the central business district and the districts number 3 and 10, the lowest for the districts number 8, 6 and 5. The investment costs have been estimated with roughly 16 million Euros.

Keywords: bicycle parking, cycling, future demand, investment costs, Vienna

1 Introduction

One of the official objectives of the city of Vienna as quantified in the Transport Masterplan 2003 is to increase the share of cycling to 8% by the year 2020 [1]. On the 15th of November 2010 the city government formulated a new, more ambitious goal of 10% share of cycling by 2015. Investments into cycling infrastructure are the key to success. Besides cycling paths and lanes the necessary infrastructure also includes safe and secure parking facilities. As international experience shows, high shares of cycling in combination with local demand concentrations can lead to significant problems with bicycles parked in public spaces. Appropriate bicycle parking facilities are needed at primary locations (home) as well as secondary locations (work, shopping, leisure, etc.). The Research Center for Transport Planning and Traffic Engineering, Vienna University of Technology (VUT), recently carried out two different studies concerning the assessment of the demand for bicycle infrastructure. A study about future requirements concerning quality and quantity for private and public bicycle parking facilities named ARNIKA (Anforderungen eines steigenden Radverkehrsanteils an die Qualität und Quantität von Fahrradabstellanlagen – Nachfrage, InfrastrukturKosten und Akzeptanz – Requirements for future quality and quantity of bicycle parking infrastructure to facilitate an increasing share of cycling – demand, investment costs and acceptance) was commissioned by the Viennese Environmental Advocacy Office (Wiener Umweltanwaltschaft) [2]. Another study dealing with the demand for bicycle parking facilities in the urban development area Seestadt Aspern was commissioned by the development agency Wien 3420 Aspern Development AG [3-5].

The starting point is an analysis of the legal framework for on- and off-street bicycle parking in Austria. Existing planning guidelines are compared with international examples from countries and cities with very high shares of cycling. Citywide data about the location of public bicycle stands are analysed. Six case study areas in the city centre, the inner city area, the suburbs and at a main railway station have been defined. Occupation rates of the bicycle stands in these areas have been counted and analysed. A web based survey has been carried out in order to gain data about bicycle parking at private locations (home, workplace). The spatial distribution of the future levels of cycling has been estimated using three different methods.

2 Legal framework and guidelines

On-street bicycle parking is regulated by the Austrian law in the road traffic regulations (Straßenverkehrsordnung – StVO). Bicycles have to be parked in a way that they do not topple or disturb the flowing traffic or pedestrians (StvO §68). Bicycling parking on a sidewalk is only allowed if it is at least 2.5 meters wide. Obstructively parked bicycles can be removed by the authorities without warning (StVO §89a).

The requirements for indoor bicycle parking in new buildings are regulated in the building codes of the federal provinces. The Viennese building code (Bauordnung für Wien – BO für Wien) is quite vague concerning the number and accessibility of bicycle parking facilities. Bicycle parking in staircases and other public parts of residential buildings is not allowed unless it is explicitly mentioned in the rental agreement [6]. Disregard can result in action of trespass. Reference values for the number of bicycle parking spaces for different types of buildings are defined in the Austrian guidelines and standards for traffic planning [7]. Residential buildings should provide a minimum of 1 bicycle parking space per 50 m² gross floor space, retail shops should provide a minimum of 1 bicycle parking space per 25 m² sales floor, etc. Nevertheless these guidelines concerning bicycle parking are not legally binding.

3 Surveys

3.1 Public space

In 2011 Vienna offered 3,426 public bicycle parking facilities with a total of 32,445 individual parking spaces [8]. The dominant system is the so called “Wiener Bügel” (Viennese hoop stand). Within the project ARNIKA surveys about bicycle parking in public spaces have been carried out in six different case study areas. These represent different characteristic types of built environments and land uses (Table 1).

Table 1 Case study areas survey bicycle parking in public spaces.

Case study area	Explanation
City centre	1st district; workplaces are dominant, important destination, roofed and non roofed facilities, time of day fluctuation
Inner city	7th district; residential use is dominant, narrow streets, focus on parking aside official parking facilities, legally and non legally parked bicycles
Inner suburb	16th district; historical suburbs outside the second ring (Wiener Gürtelstraße), residential use
Periphery	22nd district; suburb north of the river Danube, metro station with bike and ride facilities
Vienna University of Technology (VUT)	4th district; university, important destination, effect of holiday season
Western railway station	15th district; major railway station with regional, national and international trains, bike and ride, inner city shopping centre, time of day fluctuation

The observation of the occupation of public bicycle stands leads to the following main conclusions:

- The occupancy rates of public bicycle stands vary widely (Figure 1). Sometimes facilities with very low utilisation can be found quite near to facilities with utilisation close to or even above their capacity. The most important factor explaining the acceptance of a facility is the concrete micro-accessibility. The willingness to park at a facility steeply declines with the distance from the final trip destination. This behaviour could be observed most clearly at the Western railway station. During the day the bicycle parking facility next to the platforms always ran full before the other nearby facilities were accepted. The same behaviour could be observed at the case study areas city centre, periphery and VUT.
- If no bicycle parking facility is available directly at their destination then cyclists tend to park their bicycles on the sidewalk locking them to traffic signs or other suitable items. In the case study area inner city about a third of the observed bicycles was not parked at official facilities. In total about 10% of bicycles were parked illegally on sidewalks smaller than 2.5 meters wide thus obstructing the way of pedestrians.
- Roofed facilities are more attractive than non roofed ones. During the day the roofed bicycle parking facility in the city centre always ran full before the other nearby facilities were accepted. The same behaviour was observed at the western railway station.
- Bicycle theft is an important issue in Vienna. More than 90% of the observed bicycles were locked to some fixed object, a hoop stand, a traffic sign, a fence, etc.
- The occupancy rate of public bicycle stands in residential areas tends to be significantly lower than in commercial, workplace oriented areas. For more details see also section 3.2.

3.2 Web based questionnaire

The purpose of the web based survey carried out in the project ARNIKA was twofold. On the one hand it was meant to collect data about bicycle parking in private spaces and on the other hand to collect data about motives and levels of satisfaction. The questionnaire was answered by a total of 342 persons. The sample is not representative, e.g. the share of academics is much higher than the Austrian average. Some of the key findings from the web based survey are:

- At home only a minority of 6% of the Viennese respondents is parking their bicycles in the street. The majority of about 28% has access to a bicycle room within the building. About 17% park their bicycles in courtyards, 15% use compartments in the cellar, 10% park (illegally) in staircases or other general parts of the building and 8% take their bicycles with them into their flat. This illustrates that aforementioned fact that public bicycle parking facilities are less important in residential than commercial and/or workplace areas. The highest

level of satisfaction was reported by the ones who can park at home in easily accessible bicycle rooms (about 85% were very satisfied or satisfied). The lowest level of satisfaction was reported by the ones who have to park on-street or in staircases (20% and 10% were very satisfied or satisfied respectively).
- At the location of the workplace a majority of about 60% reported that the bicycle is parked in the public space either at a bicycle parking facility or locked at traffic signs etc. (Figure 2). After all 10% of the Viennese respondents take their bicycle with them into their office room. Only 5% can park their bicycle in a bicycle room in the building. Again the highest level of satisfaction was reported by the ones who can park at home in easily accessible bicycle rooms (about 80% very satisfied or satisfied). The lowest level of satisfaction was reported by the ones who have to park on-street at traffic signs etc. (about 10% very satisfied or satisfied).
- The longer the duration of stay at a destination the more important is the quality and the protection against theft.
- About a fifth to a quarter of the ones who are not already cycling every day state that they would cycle more if better parking facilities would be available at home, the workplace and shops.

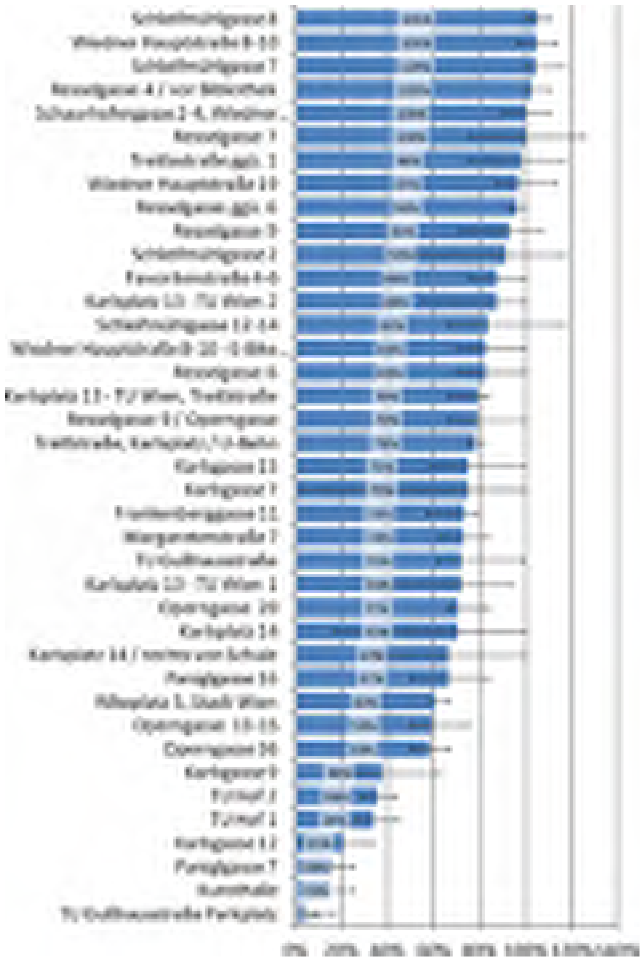

Figure 1 Average occupancy rate of bicycle parking facilities in the case study area VUT during lecture period

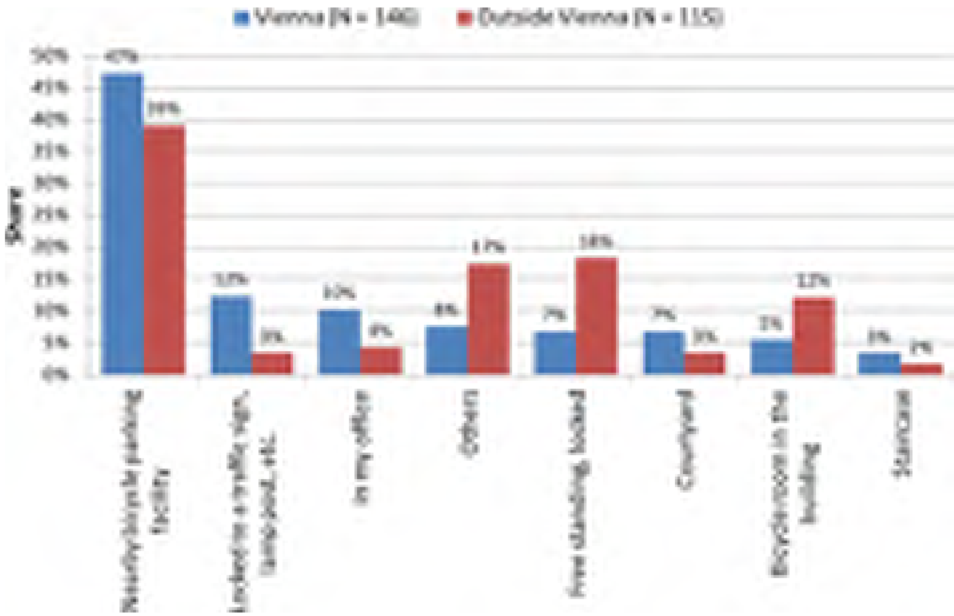

Figure 2 Type of bicycle parking at the workplace location

4 International examples

A literature review about bicycle parking in cities with very high shares of cycling has been carried out. All these cities experience certain levels of bicycle parking capacity problems. An important topic for cities with high shares of cycling are permanently parked, abandoned bicycles. Many of them develop information campaigns and special strategies for bicycle removal. The Swedish city of Malmö employs students for a systematic identification of problem areas. A direct comparison between the bicycle parking situation at a university in Copenhagen and at VUT was made. In the wider surroundings of the VUT buildings Karlsplatz, Freihaus and Gußhausstraße a total of 640 individual bicycle stands was counted at public facilities. All facilities are outdoors and not roofed. At the IT-University Copenhagen 565 individual bicycle stands have been counted, of which 550 are situated in the cellar of the university building while 15 are outdoors. While there are about 4 bicycle parking stands per 100 students at VUT there are about 26 bicycle parking stands per 100 students at IT-University Copenhagen.

5 Future demand

The attractiveness of cycling will always differ for different parts of the city. E.g. the topography of the western districts of Vienna is hillier than that of the other districts. Fitness is generally declining with old age. Hence the attractiveness of cycling also depends on the socio-demography of an area. Therefore it is unrealistic to assume that the share of cycling will ever be uniform in all parts of Vienna. Data from the census commuting statistics and household surveys have been used to estimate spatially differentiated future shares of cycling and the resulting demand for additional public bicycle parking stands. The comparison with the data about existing bicycle parking facilities results in the estimates for the future demand (Figure 3).

Figure 3 Future demand additional bicycle parking stands by registration district

6 Conclusions

According to the results of the research carried out within the project ARNIKA a total of about 44,000 to 56,000 additional public bicycle stands are needed to accommodate the intended increase to a 10% share of cycling trips. The highest demand has been identified for the central business district and the districts number 3 and 10, the lowest for the districts number 8, 6 and 5. The total investment costs have been estimated with roughly 16 million Euros. If the additional infrastructure is built in form of a five year investment program this would mean about 3 million Euros per year. The investment could be financed using revenues from car parking charges which are earmarked to be spent in the transport system. The necessary costs for the construction of public bicycle parking infrastructure would account for about 5% of the car parking charge revenues. A main result of the research presented here is that micro-accessibility is the dominant factor for the acceptance of bicycle parking facilities. Thus careful planning at the local level is essential. Hence a continuous monitoring system for public bicycle infrastructure was suggested.

Acknowledgements

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References

[1] Vienna City Administration, Transport Masterplan Vienna 2003 – Abridged Version2003: Stadtentwicklung Wien, Magistratsabteilung 18.

[2] Pfaffenbichler, P., Niel, F.: Anforderungen eines steigenden Radverkehrsanteils an die Qualität und Quantität von Fahrradabstellanlagen – Nachfrage, InfrastrukturKosten und Akzeptanz, 2013, im Auftrag der Wiener Umweltanwaltschaft: Wien.

[3] Frey, H., Brezina, T., Schopf, J.M.: Aspern Seestadt Step 2 – Anzahl und Qualität von Radabstellanlagen, Bericht zu 3.2.1, 2009, Technische Universität Wien, Institut für Verkehrswissenschaften, Forschungsbereich Verkehrsplanung und -technik: im Auftrag der Wien 3420 Aspern Development AG.

[4] Frey, H., Brezina, T., Schopf, J.M.: Aspern Seestadt Step 2 – Radabstellräume, Bericht zu 5.3.11.i, 2010, Technische Universität Wien, Institut für Verkehrswissenschaften, Forschungsbereich Verkehrsplanung und -technik: im Auftrag der Wien 3420 Aspern Development AG.

[5] Frey, H., Brezina, T., Schopf, J.M.: Aspern Seestadt Step 2 – Fahrrad und ÖV, Bericht zu 1.5.1, 2010, Technische Universität Wien, Institut für Verkehrswissenschaften, Forschungsbereich Verkehrsplanung und -technik: im Auftrag der Wien 3420 Aspern Development AG.

[6] Eichinger, A.: Abstellen von Fahrrädern im Stiegenhaus. 2003 6.11.2012]; Available from: http://www.argus.or.at/info/rad-und-recht/abstellen-fahrraedern-im-stiegenhaus.

[7] ÖFSV, Organisation und Anzahl der Stellplätze für den Individualverkehr, RVS 03.07.11 Merkblatt: Nebenanlagen – Parkplätze – Organisation und Anzahl der Stellplätze, in RVS – Richtlinien und Vorschriften für den Straßenbau2001/überarbeitet 2008, Österreichische Forschungsgemeinschaft Strasse und Verkehr; Arbeitsgruppe "Stadtverkehr; "Arbeitsausschuß "Verkehrsplanung und Raumnutzung im städtischen Bereich": Wien.

[8] MA 46, Fahrradabstellanlagen, Standorte, Wien, 2012, MA 46 – Verkehrsorganisation und technische Verkehrsangelegenheiten: Wien.

TEN YEARS OF BIKE-SHARING IN VIENNA – AN EXPLORATION INTO SUBJECTIVE USER CHOICES

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Abstract

Bike-sharing is becoming increasingly popular nowadays in every corner of the world. Vienna has one of the longest traditions of bike-sharing. The current ICT-based system, Citybike Wien, has been in service since 2003. The scheme is becoming popular and it is reported that ridership has been increasing rapidly in recent years. We have carried out a survey on user responses to this bike-sharing scheme in Vienna in spring of 2013. The results show that approximately 90% of Viennese people are aware of the bike-sharing scheme. The paper shows an overview of the Viennese bike-sharing scheme including the details of the survey results. After that, we discuss how to make bike-sharing systems more attractive to a wider range of the population and summarize suggestions for new implementation in other places.

1 Introduction

The city of Vienna has a long tradition of operating a bike-sharing scheme (BSS). Citytbike in Vienna is one of the first ICT-based urban bike-sharing systems in the world. It is in service since 2003. Users check out and in at the fixed stations with a touch-panel terminal with payment function employing debit and credit cards. It is operated by an advertisement company Gewista (a subsidiary of JCDecaux) and it is the predecessor of well-known Vélo'v in Lyon and Vélib' in Paris, which employs the same concept. At the time of writing, Citybike Wien operates 116 stations with 1,400 bicycles [1]. In 2013 790,084 rents were made, resulting in an annual average of more than 2,000 per day. By the end of 2013 Citybike Wien had a total of 482,687 registered users.

2 Methodology of the case study

The data used for this case study was collected through a CATI (computer-assisted telephone interview) carried out in April and May 2013 within the COMPASS project (co-funded by the European Commission within the Seventh Framework Programme). The telephone survey was made with individuals over 15 year-old living in Vienna. The total number of respondents is 252 and they represent the actual population distribution of Vienna well in terms of age and sex.

3 Results of the survey

The survey reveals that 66% of the respondents own a bike in Vienna as shown in Fig. 1. The bicycle ownership is in line with other surveys carried out in recent years.

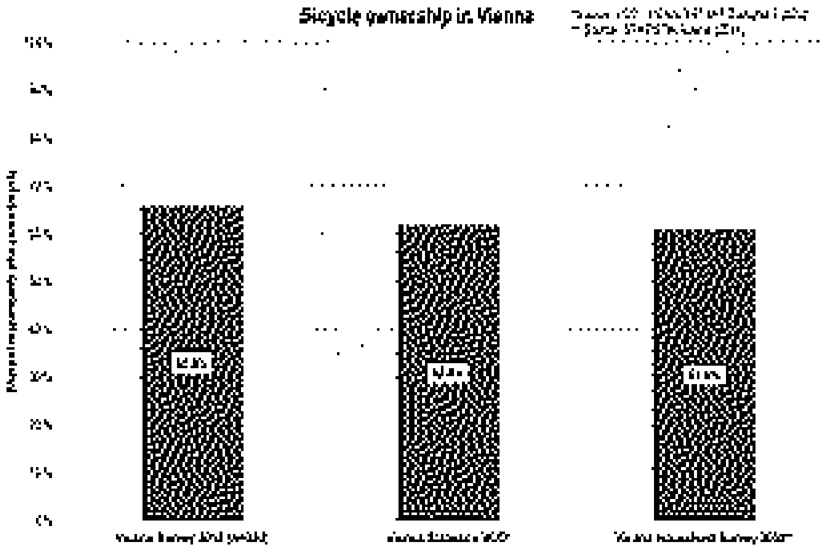

Figure 1 Bicycle ownership in Vienna among respondents and data from other surveys [2], [3].

The awareness of Citybike among Viennese people against other bike-sharing schemes (LEIHRADL nextbike, which is running in the surrounding regions of Vienna, Freiradl, which ceased operating and Call-a-bike, which is run by DB Bahn in many German cities) is already very high with 9 out of every 10 respondents being aware of it (Fig. 2). Furthermore, the high level of awareness is shown in all surveyed age bands.

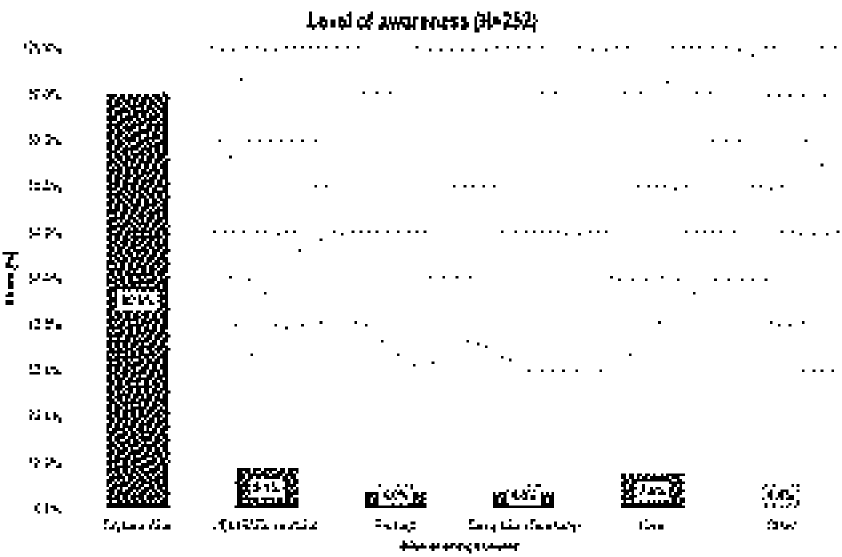

Figure 2 Level of awareness of bike-sharing schemes.

Visibility on the street plays the most important role to capture awareness of the population. 69% of the respondents in Vienna state that they learned about the BSS either by seeing the station or the bicycle in use on the street (Fig. 3).

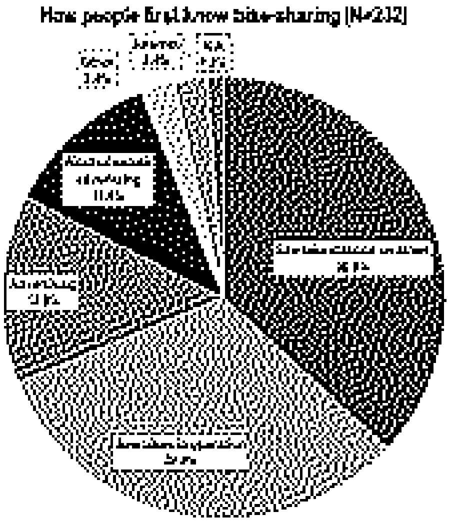

Figure 3 The way the respondents learned about bike-sharing.

Fig. 4 shows that most of the users do not use the BSS bikes often and they typically use it only once or twice a year. The shared bikes are mainly used for leisure trips. Despite such a large proportion of leisure trip usage, about 6% in Vienna have used the shared bike for commuting to and from work respectively. This implies that the BSS is used as an alternative transport mode for daily travel among some users.

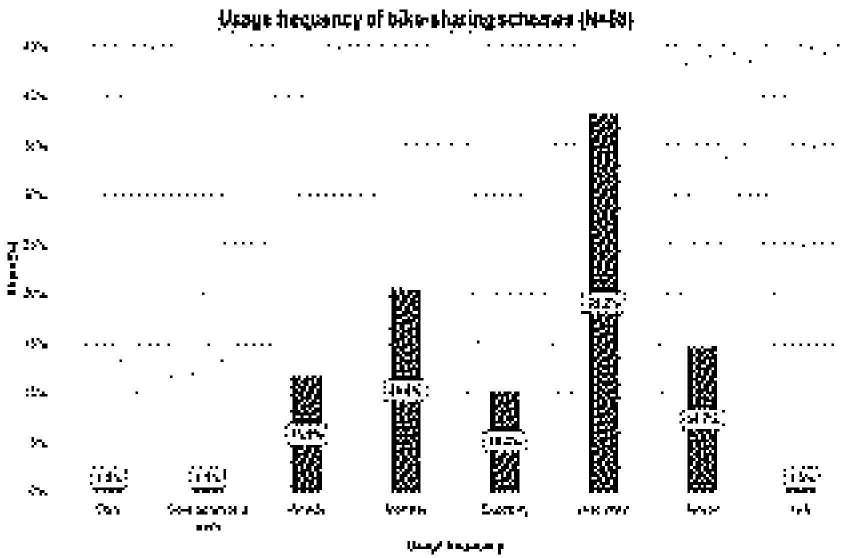

Figure 4 Usage frequency of bike-sharing schemes.

In the survey, the respondents were asked to indicate what they think as the advantages of bike-sharing. Convenience, favourable (cheap) price, and environmental friendliness are often selected as the advantage, while a certain number of respondents also point outs simplicity of usage, cycling being healthy, as well as speed advantages over other transport modes and fun (Fig. 5).

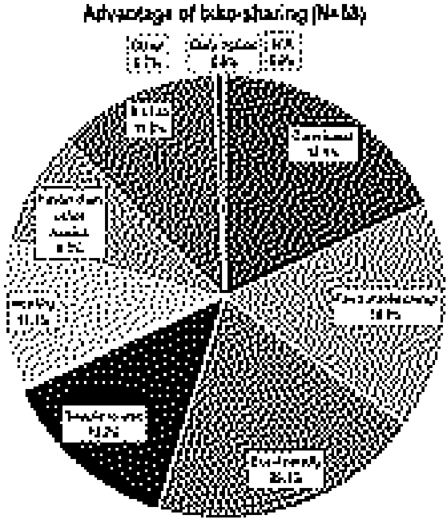

Figure 5 Advantages of bike-sharing identified by respondents.

The typical willingness to pay is around € 0.5 to € 1 per hour and thus the current pricing seems appropriate (Fig. 6 and Tab. 1).

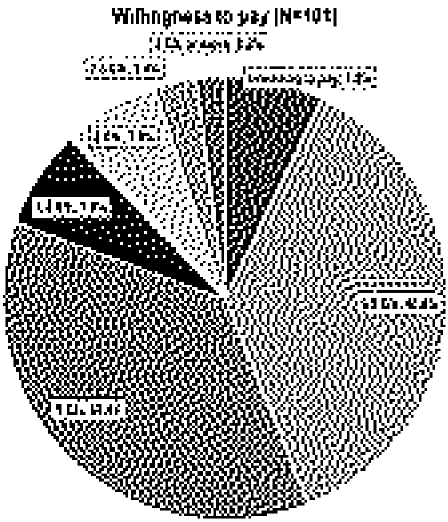

Figure 6 Willingness to pay for the shared bike.

Table 1 Rates of Citybike Wien [4].

Usage period	Tariff
First hour	Free of charge
Up to 2 hours	EUR 1 for the commenced hour
Up to 3 hours	EUR 2 for the commenced hour
Up to 4 hours	EUR 4 for the commenced hour
Longer use (up to 120 hours)	EUR 4 for each commenced hour

Many people will accept automated booking/identification methods. Especially, card-based (bike-sharing card, debit card) or phone-based identification is preferred (Fig. 7).

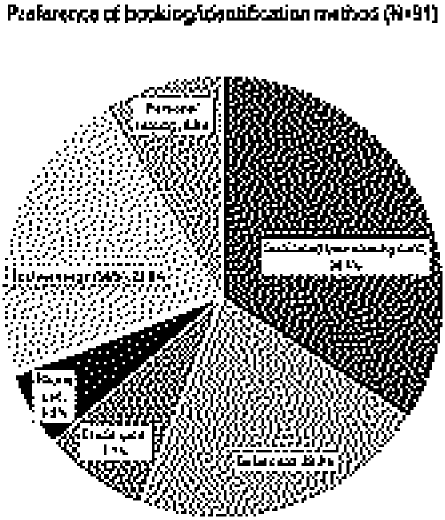

Figure 7 Preference on different booking/identification methods.

Further development of reporting broken bicycles by users (e.g. through mobile apps), mobile apps indicating general conditions of bicycles and short-term reservation (especially when there are only a few bikes available at the station) are found useful by the users. Among non-users, between one and two thirds find themselves as potential users of BSS. However, they require denser networks of the stations, easy booking system, and better bicycles to be shared (Fig. 9).

Figure 8 Willingness to use among non-users.

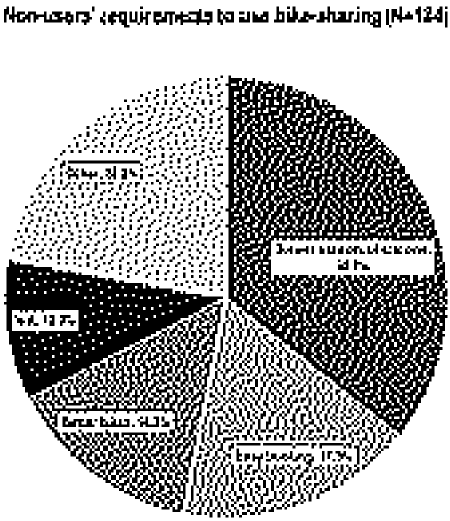

Figure 9 Non-users' requirements to use shared bikes.

4 Implementation costs and administrative burden

The OBIS handbook [5] on bike-sharing schemes quotes implementation costs for schemes from 2,500 Euro up to 3,000 Euro per shared bike. The investment costs for the construction of a new station in Vienna amount to about EUR 50,000 up to EUR 70,000 according to another source [6]. Austria's second largest city Graz (265,318 inhabitants), is planning to introduce the same BSS as in operation in Vienna in 2014 and the city estimates that between 500,000 and 1,000,000 Euro for 30 bike-sharing stations will be needed as the initial costs [7]. The information on operational costs of Citybike Wien is limited, but costs are estimated to EUR 25,000 per station. The handbook states running costs in this model per bike from EUR 1,500 to EUR 2,500. This BSS model including Citybike Wien is typically funded by the advertisement on the shared bikes as well as on other public surfaces. It should be noted that the size of the city and the metropolitan area does matter to make it feasible. The literature cited above mentions that this model is less applicable for communities smaller than 100,000 inhabitants as it tends to be difficult to achieve a critical mass of users as well as income from advertisement. Since the initial investment costs are high, decision making among the stakeholders (politicians, planners, municipalities, etc.) before the implementation can be a complex process. In dense urban areas, the arrangement of renting station locations may incur complicated negotiations with other stakeholders with right of use. This happens typically when on-street parking needs to be converted to rental stations. In addition, for successful implementation, coordination with other urban and regional transport networks is inevitable – this will require an administrative process related to transport planning. Besides minor legal aspects such as zoning of public space, and liability and contractual issues (for details, exit clause etc.) there are no legal obstacles to the implementation.

5 Conclusions and comments on transferability

The survey results show that bike-sharing is well-known among the population while those who actually use it appear to be limited. Among actual users, the leisure is dominant as trip purpose, while certain a proportion of the people in Vienna uses it as an alternative transport mode for daily trips. The current inexpensive pricing appears to be well accepted. As the certain proportion of the respondents still prefers, the cash payment system needs to be well considered to acquire more potential users although the current system using debit or credit card has to be kept to meet the needs for those who wish to use them. The preference for identification method is not uniform and there are two clear groups – one that prefer phone-based identification and another one that prefers card-based identification. This implies that providing several identification methods for the same system will meet the diversifying preference by the users.

Regarding the future development, there are two important development trends identified to capture more potential users. The first is to increase the stations density and/or extended the system so that more people can be covered within a reasonable catchment area. This will eventually help to increase the awareness through on-street visibility of the bicycles. Such enriched "hard" infrastructure for the bike-sharing will capture more potential users.

The second important future development should be based on ICT, namely short-term reservation, to make the users guaranteed with a bike available at a station nearby. Additional development is expected by diversifying identification methods especially with phone and cards (designated card/bank/credit), further easy booking/identification systems, as well as smartphone apps for showing availability and conditions of bicycles and to report broken bikes. Various BSS need to be integrated in regard to booking/identification, so that users with accounts at one BSS will be recognised automatically by other schemes.

Regarding transferability, it has to be mentioned first that similar bike-sharing schemes are already in operation in various cities including Paris, Lyon, Brussels, Dublin, Vilnius, Ljubljana, Brisbane (Australia), Kazan (Russia) and Toyama (Japan). Thus it appears that bike-sharing has good transferability for urban areas. There are several important framework conditions that should be achieved for a successful implementation of bike-sharing schemes: (1) good cycling infrastructure must exist, (2) the climate and topography must be suitable for the bicycle usage, (3) the BSS itself and bicycle-relevant traffic regulations are designed in a way so that users are identified quickly, and (4) the user interface is simple by means of card-based or phone-based identification. If these preconditions are not met, the transfer may fail.

References

[1] Gewista Pressenews: Citybike Wien – Jahresbilanz 2013, www.citybikewien.at/cms/dynimages/mb/files/PA_Citybike_2013.doc, 10.02.2014

[2] VCÖ: Österreicher besitzen mehr als sechs Millionen Fahrräder – Fahrrad braucht nach Winter ein Service, http://www.vcoe.at/de/presse/aussendungen-archiv/details/items/vcoe-oesterreicher-besitzen-mehr-als-sechs-millionen-fahrraeder-fahrrad-braucht-nach-winter-ein-service-06032013, 05.02.2014.

[3] STATISTIK Austria: Ausstattungsgrad der Haushalte – Bundesländerergebnisse, http://www.statistik.at/web_de/static/ausstattungsgrad_der_haushalte_-_bundeslaenderergebnisse_059000.pdf, 05.02.2014.

[4] Citybike Wien: Rates, http://www.citybikewien.at, 05.02.2014.

[5] OBIS: Optimising Bike Sharing in European Cities – A Handbook, p. 26, 2011.

[6] Von Sassen, W.: Öffentliche Fahrradverleihsysteme im Vergleich – Analyse, Bewertung und Entwicklungsperspektiven, Trier, pp. 99-116, 2009.

[7] Der Standard: Graz soll Leihradsystem nach Wiener Muster bekommen, http://derstandard.at/1373513991538, 05.02.2014.

BICYCLE TRAFFIC IN THE CITY OF OSIJEK

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Abstract

The city of Osijek is the fourth largest city in Croatia, with an area of about 171 km^2 and more than 107 000 inhabitants. It is located along the right bank of the Drava River whose flow defined the shape and development of the city. Namely, Osijek is positioned longitudinally along the Drava River and got a characteristic elongated shape which also defined development of the city traffic. Consequently, most of the city's main traffic routes are positioned longitudinally, along the larger residential areas and the transversal connections are intermittent by town squares and public rail route that runs through the city. In addition to public transport, city of Osijek has developed bicycle transportation. Since City of Osijek is situated in east-Croatian lowland, it has very favourable natural predisposition for the development of this form of transportation. Systematic thinking about bicycle traffic and infrastructure started in middle 1990s and few years later, first bicycle traffic counting was conducted. These days, with more than 30 km of bicycle paths and being part of international bicycle routes, Osijek is turning into a city of bicycles. Movement towards promotion of biking as fun and healthy alternative way of travel comes from different associations in Osijek. City government has also recognized the value of bike paths, which can be seen in continuous construction of new paths in different parts of Osijek and instalment of bicycle sharing system. In this paper main characteristics of bicycle paths network in the city of Osijek will be shown. Level of built of urban spaces presents a problem when planning new bike paths, therefore characteristics of bike paths vary depending on location. Special emphasis will be put on critical points of bicycle path network and comparison of bicycle paths with their presentation in Croatian legislation will be given.

Keywords: *bicycle traffic, historic overview, infrastructure characteristics, legislation, city of Osijek*

1 Introduction

Increase in number of personal cars causes a number of negative consequences. In addition to causing traffic jams and increasing the time of travel, high levels of motorizations reduce the quality of life through noise pollution, emission of toxic gases and deterioration of climate conditions. So, many governments resorted to promoting alternative modes of transportation and developed programs that aim to reduce the share of passenger cars in everyday travel. Studies have shown that increasing the use of bicycles as an alternative means of transport, on top of public transport and walking, may significantly reduce traffic congestions as well as noise pollution and toxic gases [1, 2]. For a shorter trip distances, bicycles can compete with public transport [3]. Also, it has been shown that besides being environmentally friendly, riding bicycles has a positive impact on the health of the individual. Cycling is an active mode of transportation and thus is a form of exercise, which means it has a positive effect on reducing heart diseases, high blood pressure and many other effects of passive lifestyle. Researches show that reasons for increase in bicycle usage can be found in the desire for a lifestyle change, increased awareness of bicycle importance, city administration that invests in

infrastructure and promotion of bicycles [3,4], better integration with public transit and higher number of bicycle parking lots [2]. The increase of cycling entails the issue of cyclists' safety in mixed traffic. Research has shown that actions aimed at promoting cycling and walking have a positive outcome on improving the safety of cyclists and pedestrians because of bigger number of cyclists and higher awareness for their safety [5]. After human error, deficiencies in infrastructure causes a large number of accidents [6]. Therefore, a comprehensive approach, consisting of adequate infrastructure, programs to promote cycling, limitation of car usage and appropriate urban planning is needed [7]. In order to detect and eliminate safety issues, review and design of infrastructure should be conducted through method called Context Sensitive Design, which aims to merge the function of traffic with its location [6].
In the last few years there is a substantial increase in bicycle traffic in city of Osijek. In addition to the recession and higher fuel prices, better interconnection and expansion of the bicycle paths network caused higher share of bicycles in everyday travel. Quality infrastructure is the basis for safe bicycle traffic, but the construction of the infrastructure is time consuming process which has difficulty in following an increase in bicycle traffic. In this paper, infrastructure characteristics encompassing all cycling traffic in the city of Osijek is revised and compared with existing legislation.

2 Croatian legislation on bicycle traffic

Planning, design, traffic and safety on bicycle paths should be defined and regulated by laws and regulations to ease the design and construction, to have it well and uniformly done and to ensure a comfortable and safe traffic for the end users. In Croatia, several laws and regulations mention bike paths and bike traffic, but very sporadically without giving clear guidelines. Both Roads Act [8] and the Law on Road Traffic Safety [9] define the bike paths and bike lanes. "Bike lane is part of the roadway intended for bicycle traffic, which stretches along the pavement and which is characterized by a longitudinal line on the pavement and the prescribed traffic signs." "Bicycle path is defined as traffic built area designed for bicycle traffic, which is separated from the roadway and marked with prescribed traffic signs." Regulation on basic conditions which public roads out of settlements and their elements must satisfy from the point of traffic safety [10] arising from the Law on Road Traffic Safety, stipulates the width of one lane to 100 cm and defines the traffic cross-section and clearance zone for bike paths as presented in Fig. 1. The same regulation allows the construction of bike paths along the driving lanes only if it is separated by curbs and 75cm wide road verge from those lanes.

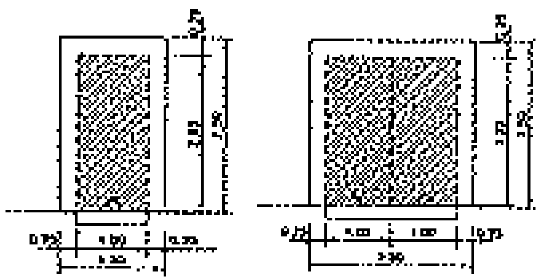

Figure 1 Clear zone and traffic cross-section for bike paths [10]

Roads Act also calls for creation of rules and regulations for the design, construction, maintenance, as well as traffic on bicycle paths but the detailed design characteristics of bicycle paths is left to the local government.
According to Urban master plan for the city of Osijek (GUP) [11], building of bike paths and parking lots is recommended, depending on urban conditions but it is not required. Also, width

for two-way bicycle traffic path of 160 cm and placing it immediately next to the pedestrian walkway is allowed, which is in contradiction with what is shown in figure 1.
When it comes to traffic safety [9, 12] only some of the rules and obligations are determined, such as rule to ride a bike on the right side of the paths or lanes, bike priority rights when in contact with a car cutting across bike lane, obligations to wear reflective clothing and use lights as well as necessary types of traffic signalizations.
A review of existing Croatian regulations shows that such regulation is either insufficient or non-existing in many infrastructure and traffic safety issues. Area of planning and infrastructure design lacks quality guidelines that would define all horizontal and vertical elements that depend on the amount of traffic and bike movement physics, position in space and interaction with other traffic participants. It also lacks guidelines on the way bicycle path are led through intersections, priority rules and rules of behaviour, speed limits, bicycle equipment and drivers' capabilities, all of which have a major impact on the safety, not only for cyclists but overall traffic.
Resulting from the increased bicycle traffic in the city of Osijek and in Croatia generally, there is a need for adequate policies which would consolidate and regulate all issues related to cycling infrastructure and traffic. City of Osijek is not the only example, infrastructure insufficiencies due to poor regulations and urban master plan can be seen in Zagreb as well [13]. Example of good legislation and its implementation in practice is found in Slovenia. Regard to the rich common history, similar mentality and civil engineering practice, this example could serve as a basis for the development of the Croatian legislation in the form of establishment of regulations and standards for the design and construction of these traffic areas.
Slovenian legislation on bicycle paths [14] holds as its main task to reduce the number of conflict points between cyclists and motor vehicles and thus increase safety and drivability. They require that the design takes into account the physical capabilities of bicycles, the number of vehicles per hour and the speed. As basic requirements for rider-friendly infrastructure singled out are: safe and comfortable traffic areas, extensive bicycle network, large number of direct links, attractive solutions and design of environment. Minimum widths for one-way traffic of 1,00 m, 2,00 m for two-way traffic and at least 2,30 m for bike paths used for recreational purposes are defined. Additional 25 cm are added on either side to get the clear zone of bike paths. Road verge should be from 0,5 to 0,75 m wide. Horizontal curve radii are also defined and should be at least 5 m (12 km/h), 8 m (16 km/h) or 10 m (20 km/h) depending on the potential bike speed. Longitudinal slopes should be 3% or less, though, they could go up to 10% but only on limited lengths. All curbs must be levelled to the ground, and manhole covers should be perpendicular to the driving direction.

3 History of cycling traffic in the city of Osijek

Situated in lowland along the river bank, with characteristic elongated shape, city of Osijek has very favourable natural predisposition for the development of cycling traffic. The beginning of cycling in Osijek dates back to the 19th century. Shortly after the first appearance of the bicycle in Croatia, the first cycling association "Concordia" has been established in Osijek [15]. It was involved in the advocacy of cycling, driver education and organization of competitions and other amusements. In 1895, first traffic timetable for bicycles in Croatia was created and drivers were obliged to take driving test, have license and register their bicycles [16]. After the First World War new cycling clubs were created and in 1927 the idea of building the first infrastructure reserved for bicycles was initiated [17]. After obtaining the land from the city, a racetrack for bikes has been built. It had elliptical shape with 400 m long trail. Later it was converted into a racecourse that could be used for motorcycle racing and a field for football matches. From the 1945 onwards bicycles become means of mass transportation. Today, there are few civic organizations promoting cycling as fun, cool and healthy way of commuting. Their work is oriented towards development of cycling traffic and safety through organizations of many events such as cycling tours and fairs, and most importantly through educations of commuters on traffic rules and behaviour in traffic. In addition to civic organizations, city administration is also promoting cycling through continuous construction of new trails and introduction of city bikes modelled on European cities. In June 2012, the project City Bike was launched and six city bikes were placed in two town squares for public use. A plan is to increase this number in a foreseeable future.

4 Cycling infrastructure in the city of Osijek

Osijek has once proudly wore the name of the greenest cities in Croatia, today it can be proud on one of the largest mileage of bike trails. Building started in 1990s with only few paths and it grew to more than 40 km of trails in the city today. On top of that there is a large number of paths to the suburbs and many are in the phase of planning and construction which creates a foundation for the use of bicycles for commuting, but also for leisure activities. Infrastructure varies greatly depending on the time of construction and spatial conditions. In accordance with the longitudinal development of the city, bike trails stretch mostly in east-west direction and are connected by a shorter cross-connecting paths (Fig.2).

Figure 2 Map of bike paths in the city of Osijek [18]

Review of current condition of cycling infrastructure was conducted by utilizing the vehicle they were built for, a bicycle. Measuring and photographing was done in order to compare existing bike paths with effective legislation and to record discrepancy and shortcomings as well as possible safety issues.

The prevailing type of bicycle infrastructure in the city of Osijek are off road bike paths. Cycling tracks as a part of mixed traffic roads are not a solution that city officials opted for. That concurs with findings [19] that bike paths are generally perceived as safer and preferable than integration with motorized vehicles. In the streets with bike paths there is generally a road verge that divides lanes for motorized vehicles from pavement for bicycles and pedestrians. However, construction details of bike paths and sidewalks differs notably.

Figure 3 Typical cross section of street with bike paths

Dominant paving material on Osijek's cycling infrastructure is asphalt which, in terms of comfort is best solution for bicycle traffic. But, there are also some paths paved by concrete pavers which, on the other hand presents aesthetically preferable solution.
Distinction between bike and pedestrian paths is done in three manners. First is delevelling and instalment of shallow curbs between two paths which was one of the initial solutions, constructed mostly in early 2000s. Outlining a yellow line is a second solution for bike and pedestrian common surfaces done recently because of spatial conditions. Some bike paths built in outer zones of the city towards the end of 2000s have concrete pavers in different colour than those of sidewalks, with yellow pavers in between. All this solutions are presented in Fig. 4.

Figure 4 Different ways of bicycle and pedestrian paths separation

Separated bike paths are rare, but those are some of the first paths constructed. They were built in the 1990s in locations where level of development was lower so there was more available space for separated bike paths.
Bike paths are mainly built on only one side of the street, thus needing to accommodate two-way cyclist traffic. They have an average width of 2 m. Running on the both side of the street, allows the bicycle paths to be narrower since in that case, they are intended for one-way traffic. However, there are some examples of insufficient path widths, with widths less than 0,8 m. In such cases, safety of bicycles as well as pedestrians is being endangered. This solutions occurs on a very busy streets and were created in the last 2-3 years because construction of bicycle paths was needed in a matter of maintaining bicycle traffic network, but any other intervention would require a lot of resources.
In intersections, bike paths are marked by red lines and there are examples of joined traffic lights for both, cyclists and pedestrians. Even though on most of the intersections bicycle paths markings are clear, the problem are discontinuities at the intersections presented in high curbs. There are some examples where high curbs are not replaced by laid curbs during adaptation of existing pedestrian paths in mixed traffic paths (usually using only yellow marking). This disrupts the drive, but more importantly it is a potential for accidents and wheels damaging. The lack of legislation is particularly seen on roundabouts where only one roundabout of total seven constructed in the city of Osijek has appropriate bicycle traffic regulation.

Figure 5 Bicycle paths design and construction irregularities in the city of Osijek

Some of the downfalls of cycling infrastructure in the city of Osijek is lack of parking facilities and even more so intrusion of different obstacles in obligatory clear zone. Bicycle parking lots are placed in front of various institutions and schools, but highly functional and safe parking lots should be placed on more public places that have high frequency of people. When obstacles are concerned a multitude of lighting poles, traffic signs, fire hydrants and trees can be seen in clear zone of bike paths. On top of that, drivers tend to park their cars, motorcycles and even lorries on bike paths.

Figure 6 Bicycle traffic difficulties in the city of Osijek

5 Conclusion

Osijek is a city that has great predispositions for a vast network of bike paths. In Osijek but also throughout Croatia it is noticeable an increase in awareness towards benefits of cycling as a healthy and practical mode of transportation. At the time of writing this article, in Croatia relevant guidelines for the design of bicycle paths did not exist, but the need for them is recognized and the Ministry of Maritime Affairs, Transport and Infrastructure is preparing Rules for the design and construction of bicycle paths. Result of this current lack of adequate regulations are various solutions for bicycle traffic on the example of City of Osijek. Therefore, it is necessary to obtain the regulations, but also to educate designers as well as all road users in order to create an environment for comfortable and above all safe traffic for all parties.

References

[1] Yang, L. et al.: Interventions to promote cycling: systematic review, BMJ, 341:c5293, 2010.

[2] Pucher, J., Dill, J., Handy, S.: Infrastructure, programs, and policies to increasing bicycling: An international review, Preventive Medicine, 50, pp. 106-125, 2010.

[3] Pucher, J., Buehler, R.: Making Cycling Irresistible: Lessons from The Netherlands, Denmark and Germany, Transport Reviews, 4, pp. 495-528, 2008.

[4] Pucher, J., Buehler, R., Seinen, M.: Bicycling renaissance in North America? An update and re-appraisal of cycling trends and policies, Transportation Research Part A, 45, pp. 451-475, 2011.

[5] Jacobsen, P.L.: Safety in numbers:more walkers and bicyclists, safer walking and bicycling, Injury Prevention, 9, pp.205-209, 2003.

[6] Dondi, G. et al.: Bike Lane Design: the Context Sensitive Approach, Porcedia Engineering, 21, pp.897-906, 2011.

[7] Pucher, J., Dill, J., Handy, S.: Infrastructure, programs, and policies to increase bicycling: An international review, Preventive Medicine, 50, pp. 106-125, 2010.

[8] Zakon o cestama, Narodne novine, br. 84/11, 18/13, 22/13, 54/13, 148/13

[9] Zakon o sigurnosti prometa na cestama, Narodne novine, br. 67/08, 74/11, 80/13

[10] Pravilnik o osnovnim uvjetima kojima javne ceste izvan naselja i njihovi elementi moraju udovoljavati sa stajališta sigurnosti prometa, Narodne novine, br. 110/01

[11] Generalni urbanistički plan grada Osijeka, Službeni glasnik Grada Osijeka, 5/06, 12/06, 1/07, 12/10, 12/11, 12/12

[12] Pravilnik o prometnim znakovima, signalizaciji i opremi na cestama, Narodne novine, br. 33/05, 64/05, 155/05, 14/11

[13] Ključarić, M., Tepeš, K., Pilko, H.: Program for development of bicycle traffic in the city of Zagreb, 2nd International Conference on Road and Rail Infrastructure, Dubrovnik, pp.899-905, 2012.

[14] Navodila za projektiranje kolesarskih površin, Novelacija, Junij 2012.

[15] Sršan, S.: Športske udruge I klubovi u Osijeku (Chapter), Povijest osječkih udruga i klubova, Povijesni arhiv u Osijeku and Gradsko poglavarstvo Osijek, Osijek, pp. 95-143, 1994.

[16] Sršan, S.: Biciklizam u Osijeku prije I. svjetskog rata, Journal for the History of sport, 94, pp.10-15, 1992.

[17] Sršan, S.: Biciklizam u Osijeku poslij I. svjetskog rata, Journal for the History of sport, 96, pp.37-42, 1993.

[18] Digitalna karta biciklističkih staza grada Osijeka, http://www.osbicikli.com/index.php/digitalna-karta, 31.01.2014.

[19] Lusk, A. C. et al.: Bicycle Guidelines and Crash Rates on Cycle Traks in the United States, American Journal of Public Health, 7, pp.1240-1248, 2013.

STUDENT BICYCLE SHARING SYSTEM IN ZAGREB –STUDOCIKL

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Abstract

The system of public bicycles in the world known as "Bicycle sharing", is a service in which bicycles are made available for shared use to individuals with or without a certain fee. Users can take and return the bike at numerous locations in the urban parts of the city. As part of the CIVITAS ELAN project (2008 – 2012), which was aimed at improving the quality of life of citizens by establishing high-quality solutions in city traffic and to promote and encourage sustainable, clean and energy-efficient modes of traffic, a pilot project of student public bicycle (Studocikl) was implemented at the University of Zagreb. In the first phase the pilot project included the Faculty of Transport and Traffic Sciences (FTTS) where is currently implemented. In the second phase the project is planned to include the remaining faculties located at the Borongaj Campus. The idea behind the Studocikl project is to connect two remote locations of FTTS, between headquarters in Vukelićeva Street and Borongaj Campus. Registration (login, logout) of users and the entire administration is done through a web portal. Time of using bicycles is in the period from 8 am to 8 pm (working hours of FTTS). Studocikl was officially put into operation on the 17 September 2012 and in this paper will analyse its one-year operation. The paper will also make proposals for further Studocikl development in terms of: installing GPS trackers for satellite bicycle tracking, the application for smartphones, full automation of the registration, and explore the possibility of extending such service and modes of transportation at all faculties in the University of Zagreb.

Keywords: sustainable urban mobility; non-motorised transport; bicycle sharing system

1 Introduction

The public bicycle system in the world known as „Bicycle sharing", represents a public bicycle rental service with or without a fare.

The Civitas Elan project funded by the European Union (2008-2012) had the objective to improve the quality of life by establishing better solutions in urban traffic, achieved by promoting and encouraging sustainable, clean and energy-efficient modes of transport. Within the project, a bicycle-sharing pilot project named Studocikl was implemented in the City of Zagreb. Such a scheme was the first of its kind in Zagreb, directed towards specific users and specific circumstances.

The partners on this pilot project were the Faculty of Transport and Traffic Sciences (FTTS) and an independent civil society named Croatian abbreviation for Sustainable Community Development (ODRAZ). This was a good example of cooperation between a civil society and the university institution.

The pilot project Studocikl was designed primarily for students and faculty staff employed at the Borongaj Campus. One of the ideas for the creation of the project was to provide students easier

transportation between two remote locations of the Faculty of Transport and Traffic Sciences. The service is completely free of charge, and maintenance costs are covered by the FTTS. Currently, the bicycles can be rented and returned after the usage at two depots. The bicycle rentals can be done with the help of staff employed at the reception of the FTTS. The rentals and disposals can be done during workdays (08:00-20:00) and Saturdays (08:00-16:00). At Sundays the service is not available. Bicycles can be used throughout the entire day, but they all have to be brought back within business hours.
This paper presents the implementation of the Studocikl project with all the predicted phases in the project. The technical characteristics, login and logout process, storage, maintenance, legal framework and the plans for the future for this project are described in detail.

2 The implementation of the Studocikl project

2.1 General

The implementation of the Studocikl project was originally conceived in two stages:
- Stage 1 – Testing (current stage):
  - The service was offered to students and the staff of FTTS. The relatively small number of users will be used to test the bicycle sharing system and to eliminate any possible problems that might occur in the stage.
  - Stage 1 was estimated to last two years (it started in 17 September 2012).
  - During the Stage 1, the preparations for Stage 2 will be done, examining the possibilities of introducing new innovations, new bicycles and bicycle parks.
- Stage 2 – extension to all institutions of the Borongaj Campus:
  - Public bicycle service will be offered to students and staff of the two remaining institutions at the Borongaj Campus: Faculty of Education and Rehabilitation Sciences and Centre for Croatian Studies.

The Borongaj Campus currently consists of three faculties under the jurisdiction of the University of Zagreb: Faculty of Education and Rehabilitation Sciences, Centre for Croatian Studies and Faculty of Transport and Traffic Sciences, with the total of 4,500 students. There are plans which include the construction of the modern campus (for eight faculties) with student dormitories and other facilities in the future.
At the moment, the Studocikl project has three basic features:20 blue bicycles with logo, two depots for bike disposals and a web portal for login and logout.
The Studocikl project offers 20 bicycles with logo. All the bicycles meet the requirements of Croatian legislation. The design is the same for each bicycle (blue colour, unisex framework, two baskets), adapted for users (in this case, students). Each bicycle is equipped with a locking mechanism, ensuring each user to meet his or her demands.
Users can get and return the bicycles at any of the two locations (depots). The first station is a small object located at the central part of the Borongaj Campus (Fig. 1). The Studocikl project was provided usage of the object by the University of Zagreb [1], and the same object was renovated within the Civitas Elan project [2]. The second station is a metal container located at the Vukelićeva Street, in front of the faculty building (Fig. 2). The usage of the object was provided by the faculty itself. Both objects are in the vicinity of corresponding buildings of FTTS, and they are monitored, secured and protected from possible thefts, vandalism and weather conditions. As a supporting entity in the Civitas Elan project, the City of Zagreb provided surface for bicycle park equipped with bike racks in front of the FTTS building in Vukelićeva Street. The bicycle park ensures the users dwelling at the faculty to easily and safely dispose their bicycles (Fig. 4).

Figure 1 Bicycle depot at Borongaj Campus

Figure 2 Bicycle depot at Vukelićeva Street

Figure 3 An example of bicycle for rental

Figure 4 Bicycle park in front of FTTS (Vukelićeva Street)

Figure 5 shows the usual routes for each mode of transport available. For each route between the bicycle stations, route length and travel time is also given. The air distance between the stations is 2.5 km. If the users travel by car, the length is 4.1 km, and the route can be crossed in 7 min. If the public transportation is used, the distance is 5.5 km crossed by 35 min. It should be noted that passengers travelling by public transport have to make a transit between tram and bus. This can raise travel time up to 50 min, which would be the same as if the shortest route was crossed on foot. Users who travel by bicycle usually need 12 min to cross the distance of 3.5 km. The Studocikl web portal is an application developed to satisfy the needs of the faculty staff related to the project. The web portal can provide the following activities:

- Continuous monitoring of bicycle depots is allowed in real-time to provide information about currently available bicycles and depot occupancy online.
- Users are allowed to create a profile on the portal. By logging into the portal, the users can make bicycle reservations for the specific time period, monitor their activities and edit their profiles.
- The users can make suggestions (e.g. locations of new stations).
- Administrators can keep track on bicycle rentals.
- Advertising is supported by social networking (Facebook, Twitter, LinkedIn, Google+, etc.).

Figure 5 Comparison of distance and travel times for each mode of transport between Vukelićeva Street and Borongaj Campus. Source: [3]

The web portal has administrator and public access. With public access, real-time monitoring of bicycle availability in depots is available, and the users have the option to log in, make reservations or to edit profile. The administrator access gives the following privileges: data downloads easy data administration (in case of involving other institutions in the future), bicycle rental monitoring, database searches, editing, or removing certain depots, bicycles or users from the database.

Figure 6 Studocikl project on the web. Source: [4]

2.2 Legal framework and terms of use for bicycles

This section represents the fundamental chapters of the Regulations on the conditions, rights and usage of public bicycle service at the University of Zagreb. The Regulations apply to future cyclists who consent to the usage of services provided by the Studocikl.
Before first use, users have to sign the Statement of the Regulations on the web portal after they have agreed on the terms and conditions clearly visible. After the signature, the users become registered in the database, and in the case of future bicycle requests they only have to sign the Application of usage. For the users that do not use the on-line application, there is an option to register with the help of the faculty staff at the reception. In that case, the hard copy of the Regulations and the Application of usage has to be presented to students in order to get their consent. The Regulations on the conditions, rights and usage of Studocikl service consists of the following chapters: General provisions, The description of the system, How to access, Obligations and limitations, Damage compensation, Privacy statements, Monitoring and protection and Transitional and Final provisions.

2.3 Bicycle rental and disposal

The process of bicycle rental and disposal includes the following:
- The users fill out an application on the web site or come in person to the faculty reception to request the service. Users have to agree on the terms of Regulations on the conditions, rights and usage of public bicycle service at the University of Zagreb only once – during the registration process.
- The employee at the reception registers the user by web portal. The confirmation of identity is carried out by the student card, student index or ID card. After the registration was successful, the user is granted the key with the specific serial number and permission to go to the depot to unlock access to the matching bicycle,
- After the usage, the bicycle has to be returned to the one of the two depots. The user then comes back to the reception (for the second time in the day) to return the key which terminates the rental process (the user becomes logged out).

2.4 Maintenance and servicing

Maintenance and servicing is a very important activity in the whole system. Servicing and maintenance is covered by FTTS. There are members of staff employed in the system, whose responsibility is to perform services, maintain the system and relocate bicycles with a van (if uneven distribution of bicycles at depots occurs). In addition to these members of staff, there are members of staff at the faculty reception who act as system administrators, allowing users to rent bicycles. During the first year of use, there were no significant bicycle malfunctions so the maintenance costs were moderate.

3 Usage statistics

After the first year of use (from October 2012 until October 2013, there were 140 registered users who had rented the bicycle at least once. The FTTS currently has about 1,450 full-time students, 710 part-time students and 179 members of staff (education personnel and others). A total number of 360 rentals were registered. Three quarters of the total number of rentals were made in order to cross the distance between the two depots (the bicycle was rented at one depot and disposed at the other). In the 60 % of the total number of rentals, the duration of the rental was less than 30 min. The average daily rental rate is one bicycle per day. However, if the winter months (due to weather), summer months (due to summer break) and weekends are excluded, the average daily rental rate then becomes 3 bicycles per day. In

October 2012, there were 95 rentals, which is the highest number of rentals in one month during the year. During the course of a year, there was no significant damage on any of the 20 bicycles in the system. Possible flaws in the data collection process might have occurred because the staff of the FTTS was introducing to the application at the beginning, resulting by incorrect login and logout process.

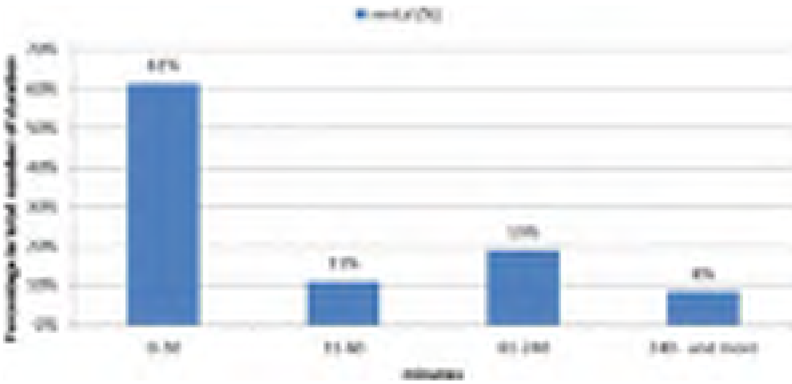

Figure 7 Bicycle rentals in relation to time of use

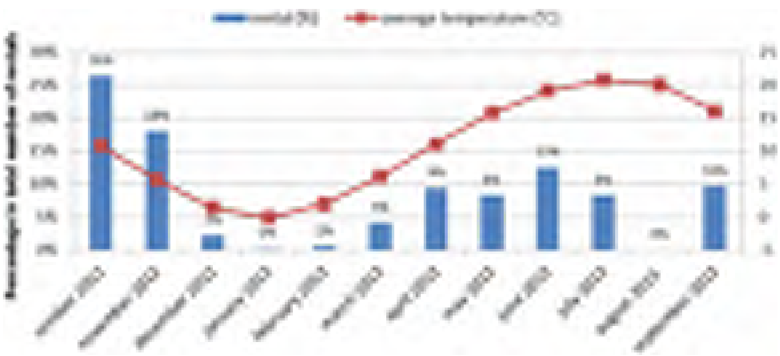

Figure 8 Bicycle rentals and average temperature in the City of Zagreb for each month in the first year of the project

4 The future development of the Studocikl project

The future development of the Studocikl project is conceived to have the following events: installation of GPS trackers (this would prevent bicycle thefts, and on the other hand this would provide the data for scientific research, such as the basic bicycle traffic parameters), introduction of smartphone application, full automation of login and logout process, introduction of electric bicycles and expansion of the service to the entire University of Zagreb. In addition, the project is encouraged to promote itself as well as the awareness of bicycle as a mode of transport in general.

5 Conclusion

The bicycle is a non-motorised mode of transport which in most cases is an acceptable solution to traffic and environmental problems in cities. Cycling contributes to good health, reduces costs and it is acceptable from the safety aspect. And because it is environmental friendly, it increases the quality of life to citizens.
The goal of the Studocikl project is not only to provide students with cheap, simple and healthy mode of transport but also to raise bicycle popularity and to offer current and potential users an alternative mode of transport, both for recreation and daily commuting.
The frequency of the usage (winter and summer months excluded) of 3 rentals per day did not meet the expectations, so the focus of the project is planned to be put on advertising in order to attract more users in the future.
Based on the experiences from the project, the possibilities for the implementation of the project to the entire City of Zagreb and other urban areas in the Republic of Croatia will be analysed. In addition, possibilities to upgrade the system with new innovations will be explored. Further development of the Studocikl project will serve as a platform for FTTS staff to conduct scientific research in the field of traffic and transport.

References

[1] "Univesity of Zagreb." [Online]. Available: http://www.unizg.hr/. [Accessed: 13-Feb-2014].

[2] "CIVITAS Zagreb." [Online]. Available: http://www.civitaszagreb.hr/naslovna/. [Accessed: 13-Feb-2014].

[3] "Interaktivna karta Hrvatske – HAK." [Online]. Available: http://map.hak.hr/. [Accessed: 12-Feb-2014].

[4] "Studocikl ." [Online]. Available: http://www.fpz.unizg.hr/studocikl/. [Accessed: 12-Feb-2014].

ANALYSIS OF PEDESTRIAN AND CYCLIST BEHAVIOUR AT LEVEL CROSSINGS

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Abstract

Level crossings (LCs) are points of conflict between rail and road traffic. Therefore, from the aspect of safety they are potentially high-risk traffic points. Traffic participants at LCs are pedestrians, cyclists, motorcyclists, car drivers and locomotive drivers. The behaviour of traffic participants represents the main cause of traffic accidents at LCs. Most research examining road users' behaviour at LCs has focused on car drivers and there are few studies dedicated to pedestrians and cyclists, especially in Croatia. Cyclists are often treated like pedestrians but cyclists can travel much faster than pedestrians, which can cause unexpected behaviour. The paper gives an overview of the existing cycling features in the City of Zagreb and the statistics of accidents on LCs in the Republic of Croatia. Also, through a review of the major recent studies on the behaviour of pedestrians and cyclists at railroad crossings, the trends have been presented as well as the results of research. The review paper will serve as the basis for further research of design, traffic safety and the behaviour of pedestrians and cyclists, and their correlation at the LCs in the Republic of Croatia.

Key words: level crossings, pedestrians, cyclists, traffic safety

1 Introduction

Level crossings (LCs) are places of direct conflict between rail and road traffic. Since these are collision points of two traffic systems, they represent from the safety point of view traffic points of high risk at which there often comes to emergency situations, sometimes with the severest of consequences. Statistical data show that in more than 90% of emergency cases the main cause lies in the road motor vehicle drivers and pedestrians. In the Republic of Croatia there is a total of 1,514 LCs out of which 60 are level pedestrian crossings. The safety level depends on the category of the railway line and road, permitted speed, field conditions and local circumstances at the crossing point. Consequently, LCs can be secured by road traffic signs (minimally the sign STOP and St. Andrew's cross) and the visibility triangle or security device (automatic device – light-audio signals with or without half-barriers and mechanical device – barriers). Automatic or mechanical devices are used at 531 LCs, whereas the remaining 923 are secured by road traffic signs and the visibility triangle [1]. The issue of LCs is included in a large number of laws, regulations and other documents defining the security method as well as under whose jurisdiction lies the solving of certain segments of LC [2, 3]. On the other hand, although cycling has significantly increased over the recent several years, the cycling issues are included in the legal regulations at an extremely low extent [4]. Neither are the investments into infrastructure improvement sufficient, which affects negatively the safe flow of traffic and leads to emergency situations.

2 Analysis of cycling traffic

2.1 Summarized overview of carried out research of cycling traffic

A significant increase in cycling in the countries worldwide, particularly in the cities, has been evident and is the result of implementing long-year structural programs and measures as a special segment of comprehensive traffic policies with the aim of increasing bicycle traffic. The majority of these programs and measures are in correlation with the implementation of measures for the improvement of other travelling modes.
Review of literature [5] suggests the need to facilitate cycling through appropriate bicycle infrastructure, integration with public transport, traffic calming, training and education programs, bicycle access programs, and legal issues. Countries and cities with high levels of cycling and good safety rates tend to have extensive infrastructure, as well as pro-bicycle policies and programs, whereas those with low cycling rates and poor safety records generally have done much less. However, it is not clear which measures are the most effective and should be given priority in designing and implementing a pro-bicycle policy package. A significant increase in bicycle traffic is evident from the following. For example, Berlin almost quadrupled the number of bicycle trips between 1970 and 2001 and doubled the bicycle share of trips from 5% in 1990 to 10% in 2007. In spite of the sharp rise in cycling, serious injuries in Berlin fell by 38% from 1992 to 2006. In only six years, the bicycle share of trips within the City of Paris more than doubled from 1% in 2001 to 2.5% in 2007. The bicycle share of trips in Bogota quadrupled from 0.8% in 1995 to 3.2% in 2006. The total number of bicycle trips in London doubled between 2000 and 2008, while cyclist injuries fell by 12% over the same period. Amsterdam raised the bicycle share of trips from 25% in 1970 to 37% in 2005 while serious cyclist injuries fell by 40% between 1985 and 2005. From 1995 to 2003, the bicycle share of trips in Copenhagen rose from 25% to 38% among those aged 40 years and older [5]. Looking at these research results one cannot determine which measures/packages dealt with the issue of cycling traffic at level crossings, both from the aspect of relevant bicycle infrastructure, improvement of cyclists' traffic safety, etc. Therefore, it is necessary to carry out further systemic research of cycling traffic at LCs.

2.2 Features of bicycle traffic in the City of Zagreb

In the City of Zagreb for the last 15 years there have been ongoing measures to improve and encourage bicycle traffic in the overall travel. In the mid 1980s the bicycle traffic and bicycle-oriented surfaces were intended exclusively for recreational and sporting purposes (the first example was the bike path around the lake of Jarun) which is mostly the case today. In the period since 2010 additional 21 km of cycling paths have been made in the wider urban area and 138 km of sport-recreation cycling paths in the Nature Park Medvednica, which is a total of approximately 370 km. Also, the City of Zagreb undertook a number of other traffic technical and regulatory interventions with the aim of improving the conditions for bicycle traffic (e.g. removal of urban and architectural barriers, marking of cycling areas with red filled (infill) lanes in the full profile, construction of bicycle path or lane during reconstruction and major road repairs). First official data regarding the volume of bicycle traffic were recorded in the year 1999 for the purpose of a traffic study of the City of Zagreb [6]. The research covered in this study shows that only 0.7% of the daily trips are realized by bicycle. However, it is interesting to note that 51% of households said that they had at least one bicycle, which represents a respectable potential for greater use of bicycles as means of travel. After the above mentioned traffic study, several measurements and surveys were conducted which provided an approximate image for certain characteristics of the intensity of bicycle traffic. In the study performed by ISIP-MG [7], measurement of traffic at 16 locations was carried out, mostly on the city's busiest traffic corridors. Based upon these limited measurements, it can be assessed that there is a certain amount of increase in bicycle traffic. Furthermore, by carrying out comprehensive research for the needs of the Project CiViTAS ELAN ZAGREB at certain locations the measurement of cycling traffic was carried out [8]. Figure 1 shows the results of measurements for 2008 and 2012. By comparing the measurement results one can conclude that at the observed locations significant increase in bicycle traffic was recorded, in the amount of 17.18% to even as much as 72.25%, although the cycling infrastructure is still insufficiently developed and not at an acceptable level.

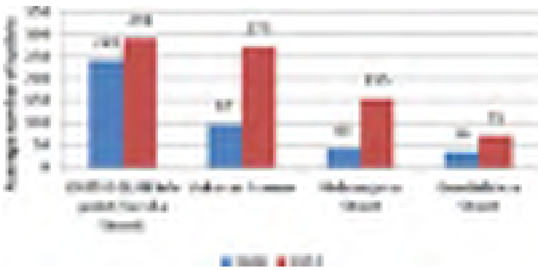

Figure 1 Average number of cyclists at four control locations [8]

Table 1. shows a significant reduction in the number of fatalities during 2012 in relation to the previous year 2011 by as much as 71.43%. The statistical reports [9] state as the most frequent causes of traffic accidents involving cyclists the following: riding across pedestrian crossing, failure to use cycling paths/lanes, riding on sidewalks, and no lights at night. It should be mentioned that during 2012 in traffic accidents involving cyclists, they were responsible for about 2/3 of traffic accidents of this type, which can be attributed to the low level of traffic culture, i.e. disregard of traffic rules. However, it is impossible to determine from the data what is the number and what are the types/consequences of accidents that occurred at LCs involving pedestrians, i.e. cyclists.

Table 1 Number and consequences of traffic accidents involving cyclists in the area of the City of Zagreb [9]

Number of traffic accidents involving cyclists	Year		Difference
	2012	2011	[%]
with fatalities	2	7	-71.43
with injured	309	297	4.04
with material damage	93	110	-15.45
Total	404	414	-2.42

Further development and improvement of bicycle traffic in the City of Zagreb will be focused upon interventions that can be defined through the following program components: improving conditions in the existing bicycle network, further development and expansion of bicycle paths or lanes, implementation of public bicycle service (e.g. nextbike), amending legislation regarding regulation of bicycle traffic, education and marketing activities to encourage people to use bicycles as a means for the realization of commuting [10]. Consequently, it is necessary to systematically monitor the movement of bicycle traffic and the safety level for the area of the City of Zagreb. Among other things, this would create a certain base of traffic data with the objective of more detailed analysis of non-motorized traffic in/at the area of LCs. Such analysis should result in a proposal of measures and guidelines for proper management and design of non-motorized traffic in/at the areas of LCs.

3 Analysis of safety situations at level crossings

In railway traffic an safety situation represents an undesired, unintentional or unexpected event or sequence of such events, which results in any kind of damage, regardless of the amount of damage. Emergency situations are divided into four basic categories: serious accidents, accidents, disturbances and avoided accidents [11]. Serious accident is an emergency situation in railway traffic in which at least one person has been killed, and/or five or more persons are physically injured, and/or the material damage is greater than five million kuna. An accident is an emergency situation in railway traffic with harmful consequences such as severe physical injuries of up to four persons and material damage that can be estimated at a value of up to five million kuna [1]. Traffic safety at LCs means safety of railway and road traffic. The safety condition at LCs in the Republic of Croatia is best shown by the statistical data about the number of traffic accidents and consequences. An analysis of accidents at LCs and their consequences in the period 2007-2008 still show a significant number of accidents, either fatal or greater number of injured persons, and with considerable material damage. Particularly worrisome trend of steady growth in the number of injured people tend to LCs with the highest level of security. Comparing the 2011 and 2012, it is evident that in the 2012 the number of serious accidents is significantly reduced (25%) and the number of fatalities as well (34.6%). In the 2012 on the LCs happened a total of 45 accidents, eight serious accidents and 37 accidents. It is disturbing the fact that seven of these accidents occurred at LCs secured with automatic devices with light-acoustic signaling and semi-barriers in which two people died and one person was seriously injured. On the LCs secured with light-acoustic signaling occurred 13 accidents in which one person was killed and eight were seriously injured, while the 25 accidents that occurred at crossings marked by road traffic signs "Stop" and "Andrew's Cross", five people were killed and six were seriously injured (Figure 2., Table 2. and 3.).
According to statistics published by the European Railway Agency (ERA), there are at least 123,000 LCs in the European Union (EU). Most of them (71%) are passive LCs without any active warning or protection devices, such as lights, bells or gates. Roughly 45% of LC accidents in the EU occur at passive LCs, and 65% of road users involved in accidents are drivers or occupants of passenger cars or heavy vehicles. In 2010, there were 359 LC accident fatalities in the EU. This represents 29% of fatalities in railway accidents but only about 1.2% of all road accident fatalities. Most of the direct causes are related to the behaviour of road users (95% [12]) such as distraction, while other causes of accidents were related to weather conditions or the condition of the driver (e.g. alcohol/drugs) [13]. An evaluation of accident data on 256 LC accidents was carried out as part of the SELCAT (Safer European Level Crossing Appraisal and Technology) project. About 91% of level crossing accidents in the EU were found to be caused by human failure, and over 80% were found to have been caused by the driver of the road vehicle not respecting the traffic rules [14].

Table 2 Overview of emergency situations at LCs [1]

Safety situation		SERIOUS ACCIDENTS						ACCIDENTS					
year		2007	2008	2009	2010	2011	2012	2007	2008	2009	2010	2011	2012
All LCs	secured with SS-devices	4	2	2	1	6	3	15	13	17	11	14	16
	secured with traffic signs	5	4	5	4	4	5	48	19	40	25	20	19
	pedestrian crossing	0	1	0	0	1	0	0	0	0	0	0	0
Total		9	7	13	5	15	8	63	32	57	36	34	37

Table 3 Analysis of consequences of emergency situations at LCs [1]

Safety situation		FATALITIES						SEVERELY INJURED					
	year	2007	2008	2009	2010	2011	2012	2007	2008	2009	2010	2011	2012
LCs	secured with SS devices	[illegible]	[illegible]	[illegible]	1	[illegible]	[illegible]	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	secured with traffic signs	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	pedestrian crossing	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	Total	12	3	15	7	15	8	12	15	[illegible]	10	8	15

Figure 2 Emergency situations at LCs in Croatia [1]

4 Overview of studying the behaviour of participants in road traffic at level crossings

Most research examining road users' behaviour at LCs has focused on car drivers, and there are fewer studies dedicated to pedestrians and cyclists. One of the significant problems is trespassing. A number of studies have suggested that the main reason for trespassing is taking a shortcut from point A to point B because the authorised route is assessed to be too far away. According to a study in Finland, most people were trespassing while going shopping, jogging, or on their way to school or work. Thirty-five percent of all respondents trespassed daily or almost daily. It is significant that 67% of all respondents answered that they trespassed at least once a week. Half of the respondents assessed that the trespassing is either completely or fairly safe. Overall, 59% of the respondents considered trespassing illegal, 15% considered it legal and 26% did not know. One of the measures to decrease trespassing is the installation of countermeasures. The data show that there were 78 bicycle trespassings before installation of countermeasures and zero trespassing after the installation of countermeasures [15]. Studies indicate that different road users might interact differently with the LC system. In particular, on-road studies indicate that the content of individuals' situation awareness or their sense of what is going on around them varies depending on their transportation mode, although some authors suggest that these differences result in cognitive incompatibilities between different road users [16]. Motorcyclists appear to be more focused on anticipating potential hazards than car drivers, whereas cyclists in dense traffic may focus more on seeking safe alternative travel routes such as bicycle lanes, service lanes and footpaths [17]. Beanland et al. designed a longitudinal survey to record interactions at LCs over a two-week period. The survey focused on understanding how individuals behaved in the presence of a train, which included examining the decision that they made (to stop or proceed before the train) and the specific factors that assisted their decision-making in that situation. The sample included 166 adults residing in metropolitan Melbourne (80%) and regional Victoria (20%), with a mix of car drivers, motorcyclists, cyclists and pedestrians. Visual information (e.g., flashing lights) emerged as one of the most influential factors for car drivers and motorcyclists, whereas pedestrians and to a lesser extent cyclists relied more on auditory information (e.g., bells) to alert them to the presence of a train. Pedestrians were also more likely than other road users to speed up and cross the tracks ahead of an approaching train. Overall, these results emphasise the importance of designing road systems to support cognition and behaviour across a range of road users, in order to ensure a safe system for all [18]. In addition, a relatively large survey of 1,862 cyclists in Queensland, Australia found that women are more likely to cycle off-road than men, and are less likely to commute by bicycle than men, and that, although factors related to traffic conditions, motorist aggression and safety are concerns for both women and men, women report a far greater number of these constraints [19]. Pedestrian treatments on risky behaviour at light rail transit LCs was researched by Siques. Five treatments were evaluated: pedestrian automatic gates, a prototype active pedestrian warning device, a prototype active "Look Both Ways" sign, barrier channelization at a skewed crossing, and a "Stop Here" pavement marking. Statistically, to reduce risky pedestrian behaviours, pedestrian automatic gates were reported as the most effective. However, pedestrians were found to be less likely to look both ways or stop before entering a crossing when a pedestrian automatic gate or pedestrian flashing light was installed. Interestingly, the "Look Both Ways" sign was found not to be effective in reducing the number of pedestrians entering the crossing immediately after train departures. Research on examples of innovative warning and control devices at LCs include four factors that enabled pedestrians to walk safely through LCs: pedestrian awareness of the crossing, existence of a pedestrian path across the trackway, pedestrian awareness of and ability to see an approaching train, and pedestrian understanding of the potential hazards at LCs [20]. Khattak and Luo [21] investigated pedestrian and cyclist behaviour at a dual-quadrant gated LC located in the residential area of the City of Fremont, Nebraska. The crossing has two sets of railroad tracks, two paved highway lanes, and is equipped with dual-quadrant gates. The gates have flashing lights, crossbuck sign and an audible bell. The crossing is equipped with a crosswalk on its west side for pedestrian use, which is sometimes used by cyclists as well. Most pedestrians and bicyclist use the crosswalk, but a few occasionally use the street to negotiate the crossing. Violations by pedestrians and cyclists were monitored using video surveillance in three instalments during the years 2008, 2009 and 2010. Violations were divided into four groups: 1) passing under descending gates, 2) passing around fully lowered gates, 3) passing under ascending gates, and 4) passing around fully lowered gates between successive trains. During data collection a total of 1,074 non-motorized individuals were observed indulging in 807 violations. On average, 1.70 individuals were observed per crossing event and 1.27 violations per crossing event were noted. Analysis showed that there were no differences in the occurrence of gate-related violations by pedestrians and cyclists. Young children of around 8 years of age or younger were involved in 25% more gate-related violations than older crossing users. Violations increased with the presence of more individuals at the crossing during train crossing events, but the contribution from young children was greater than that from older crossing users. In Holland there were 48 fatal incidents in 1985 and the government policy was to decrease this amount of LC accidents by 50% by the year 2010. In 2006 on 2,724 public LCs of all types there were 4 accidents that took 9 lives and left 11 people injured, while no derailments due to LC accidents occurred. There were 93% of accidents caused by errors of some description by road users. Of these, there were 39% conscious errors. In 53% of accidents the road user did not see the train approaching until impact. Males were substantially more involved in incidents and accidents than females, while more than 30% of those involved belonged to the age group 20-29. The total involvement of age groups peaks between 10 – 59, school and working ages [22]. Reducing pedestrian and cyclist violations at LCs will improve traffic safety, but most technology-based countermeasures (e.g., automatic pedestrian gates, electronic signs) are expensive and difficult to maintain. Other options for that are enforcement and public outreach and proper education.

5 Conclusion

The existing level of adjustment of the traffic infrastructure to current and future growth of bicycle traffic is not sufficient, which can be seen from the presented characteristics of bicycle traffic in the City of Zagreb. The safety at LCs is a complex problem. Apart from technical and technological factors also human behaviour has to be taken into consideration and this is very difficult to predict, monitor and track. Whether referring to accident which resulted from the collision of a train and motor vehicle, cyclist or pedestrian, the consequences are larger by injuries. Research showed that the main cause of collision at LCs is the behaviour of road user participants. This can be largely assigned to risky behaviour of drivers i.e. their lack of attention when driving a vehicle, disregard of traffic regulations and stress. Risky behaviour of motorists, cyclists or pedestrians at LCs is extremely dangerous and mostly results in emergency situations. The road traffic participants are often not aware of potential danger at LCs, and adaptation and response time are often related to implicit impacts on perception and capability of decision making, e.g. stress, fatigue, personal problems, and physical and mental state. The review paper will serve as the basis for further research of design, traffic safety and behaviour of pedestrians and cyclists, and their correlation at LCs in the Republic of Croatia. Finally, apart from technical and technological design solving of the crossings, the systemic activities in terms of education of road motor vehicle drivers, cyclists and pedestrians is of extreme importance, with the aim of upgrading the level of their traffic discipline, culture and awareness about the causes and consequences of risk behaviour at LCs.

Acknowledgements

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References

[1] Godišnje izvješće o sigurnosti u 2012. godini, Hrvatske željeznice, Zagreb, 2013.

[2] Pravilnik o uvjetima za određivanje križanja željezničke pruge i drugih prometnica i za svođenje i određivanje zajedničkoga mjesta i načina križanja željezničke pruge i ceste, NN 121/09, 123/12, Zagreb, 2012.

[3] Pravilnik o načinu osiguravanja prometa na željezničko-cestovnim prijelazima i pješačkim prijelazima preko pruge, NN 121/09, Zagreb, 2009.

[4] Pravilnik o funkcionalnim kategorijama za određivanje mreže biciklističkih ruta, NN 91/13, Zagreb, 2013.

[5] Pucher, J., Dill, J. & Handy, S.: Infrastructure, Programs, and Policies to Increase Cycling: An International Review, Preventive Medicine, 50, pp. 106-125, 2010.

[6] Traffic Study of the City of Zagreb, MVA, Zagreb, 1999.

[7] Izrade prijedloga nacrta pravilnika o regulaciji biciklističkog prometa i smjernica za projektiranje biciklističkih staza i traka, ISIP-MG d.o.o., Zagreb, 2011.

[8] Engels, D., Kontić, D., Matulin, M., Mrvelj, Š., Van Cauwenberge, B., Valkova, J., Vilarinho, C., Pedro Tavares, J., Van Aken, E.: CiViTAS ELAN Final Evaluation Report, ELAN Deliverable No.10.11. European Commission, Brussels, 2011.

[9] Ministarstvo unutarnjih poslova, Policijska uprava zagrebačka, Stanje sigurnosti u cestovnom prometnu na području Policijske uprave Zagrebačke, Zagreb, 2012.

[10] Ključarić, M., Tepeš, K. & Pilko, H.: Program for Development of Bicycle Traffic in the City of Zagreb, 2nd International Conference on Road and Rail Infrastructure, Dubrovnik, pp. 899-905, 2012.

[11] Pravilnik o izvanrednim događajima u željezničkom prometu, NN 64/09, Zagreb, 2009.

[12] Slovak R. & Wegele, S.: Level Crossing Accidents Statistics, Normalisation and Comparison, Proceedings of 2nd SELCAT Workshop, Marrakech, Morroco, pp. 83-100, 2007.

[13] European Railway Agency, Railway Safety Performance in the European Union 2012, European Railway Agency, Valenciennes, France, 2012.

[14] Lazarevic, N., Khoudour, L., El Koursi, E.M., et al.: SELCAT D2 – Report about Examination of Actual and Potential Technologies for Level Crossing, 2012.

[15] Silla, A.: Improving Safety on Finnish Railways by Prevention of Trespassing, Disertation, VTT, Helsinki, December 2012.

[16] Walker, G.H., Stanton, N.A., & Salmon, P.M.: Cognitive Compatibility of Motorcyclists and Car Drivers, Accident Analysis and Prevention, 43, pp. 878-888, 2011.

[17] Salmon, P.M., Young, K.Y., & Cornelissen, M.: Compatible Cognition Amongst Road Users: The compatibility of Driver, Motorcyclist, and Cyclist Situation Awareness, Safety Science, 56, 6-17, 2013.

[18] Beanland, V., Lenné, M.G., Salmon, P.M. & Stantond, N.A.: A Self-report Study of Factors Influencing Decision-making at Rail Level Crossings: Comparing Car drivers, Motorcyclists, Cyclists and Pedestrians, Proceedings of the 2013 Australian Road Safety Research, Policing & Education Conference, Brisbane, Queensland, pp. 11, 2013.

[19] Heesch, K.C., Sahlqvist, S. & Garrard, J.: Gender Differences in Recreational and Transport Cycling: A Cross-Sectional Mixed-Methods Comparison of Cycling Patterns, Motivators, and Constraints, International Journal of Behavioural Nutrition and Physical Activity, 9(1), pp. 106-118, 2012.

[20] Siques, J.T.: Effects of Pedestrian Treatment on Risky Pedestrian Behaviour, Transportation Research Record, 1793, Transportation Research Board, Washington, D.C., pp. 62-70, 2002.

[21] Khattak, A. & Luo, Z.: Pedestrian and Bicyclist Violations at Highway-Rail Grade Crossings, Transportation Research Record, 2250, Transportation Research Board, Washington, D.C., pp. 76-82, 2011.

[22] Van der Mark, P.: The Dutch View on Level Crosing Safety, The Institution of Engineering and Technology Railway Network, UK, 2007.

STUDY ON THE AVAILABILITY OF "TWITTER" DATA FOR FORECASTING SUSPENSION TIME OF RAILWAY OPERATION

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Abstract

Recently, a lot of people use Social Networking Service (SNS) to report their situation and do some findings around them. Especially, "twitter" is one of the easy ways to report some information for followers, and "twitter" users could send information and personal situations around them to "twitter" platform in any time. The amount of "twitter" data might be a lot more than the existing statistical data since some information are being uploaded at any time the users find it. Therefore, the numbers and the contents of "tweets" are different depending on the scale of the event, and a relevant key word is reported to "twitter" by the users. For example, when the railway operation stops, "twitter" users report that situation for the followers. The contents of the "tweets" are railway accident information, recovery time and complaint to railway company/staffs and so on. Then the duration of suspension time of the railway operation can be forecasted using the "twitter" data of the railway users. In this study, number and contents of "tweet" are analysed under accidental conditions which are a small railway accident and a railway accident in which a lot of people get influenced. Furthermore, the authors have developed suspension time forecasting model using "twitter" data, and the forecasted time is verified by comparing with the existing statistical data which is utilized by the Ministry of Land, Infrastructure, Transport and Tourism in Japan.

Keywords: railway, Social Networking Service, twitter, forecasting model

1 Introduction

There are many causes to delay the railway service schedule. And many passengers are assumed to be suffering a loss due to these delays. To investigate the current condition of the delay in Tokyo Metropolitan Area (TMA), statistics by Ministry of Land, Infrastructure, Transport and Tourism (MLIT) can be utilized. This statistics compiles the reports regarding railway accident and trouble with more than 30 minutes delay. According to this statistics, we can know when the accident happened, what the cause was, how large the influence was, and etc... According to the statistics that the transition of the number of railway accidents. It had been decreasing until 2003. The reason is that MLIT devised measures to reduce the number of railway accidents. For example, MLIT indicated railway companies to install emergency stop buttons in platforms, fall detection mats into railroads, and evacuation spaces under platforms [1],[2].
Recently, a lot of people use Social Networking Service (SNS) to report their situation and do some findings around them. Especially, "twitter" is one of the easy ways to report some information for followers, and "twitter" users could send information and personal situations around them to "twitter" platform in any time. For example, when the railway operation stops, "twitter" users report that situation for the followers. The contents of the "tweets" are railway

accident information, recovery time and complaint to railway company/staffs and so on. Then the duration of suspension time of the railway operation can be forecasted using the “twitter” data of the railway users. In this study, number and contents of “tweet” are analysed under accidental conditions which are a small railway accident and a railway accident in which a lot of people get influenced. Furthermore, the authors have developed suspension time forecasting model using “twitter”.

2 DATA

2.1 Tweet Data Collection

Table 1 shows the twitter data which are created by NTT DATA Corporation. The data has created date and tweet text data in Japanese. Authors get tweet data regarding railway accident using key words which are “delay”, “stop”, “situation” and so on. Conditions of accident selection are date, time, scale of accident and modality of accident. Ten railway accidents were selected for this analysis as shown in table 2. An average suspension time is about 53.9 (minutes).

Table 1 “Twitter” data (in Japanese)

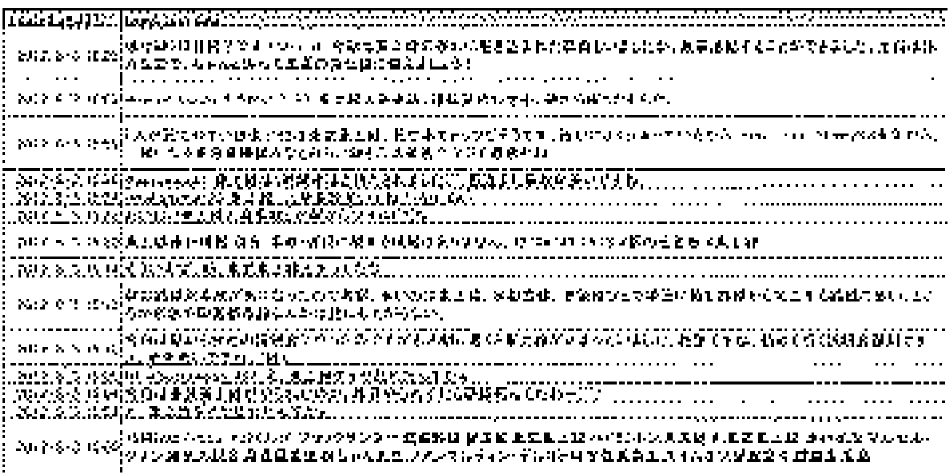

Table 2 Selected railway accidents

Case	Year	Month	Day	Duration of suspention time (minutes)
A	2012	April	25	10
B		June	13	51
C		July	10	58
D		August	3	93
E		September	6	66
F	2013	January	27	72
G		February	6	105
H		February	13	14
I		February	20	40
J		March	5	30

2.2 Basic analysis

Figure 1 shows the changes with the lapse of time of number of “tweet”. The “tweet” increase from occurrence of railway accident to 60 minutes in each cases. During the railway operation stopped, railway users send information and personal situations around them to “twitter” platform.

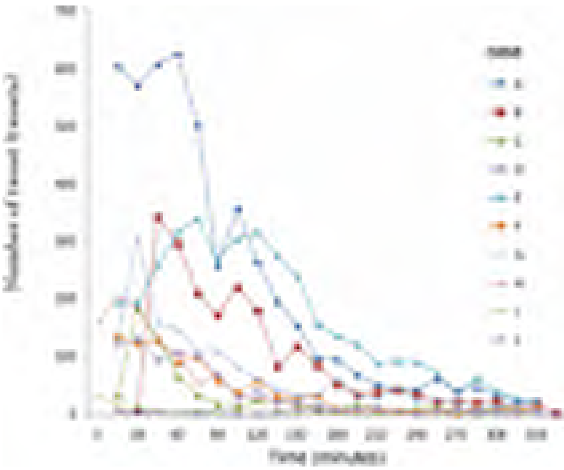

Figure 1 Changes with the lapse of time of number of “tweet”

Figure 2 shows changes with the lapse of time of accumulation of number of “tweet”. After the accident occurred, the number of “tweets” is stable in 150 minutes. Average suspension time of railway accident is about 53.9(minutes), but the “tweet” were sending to twitter platform after the railway operation start. This phenomenon means the delay of the railway affects it for a long time.

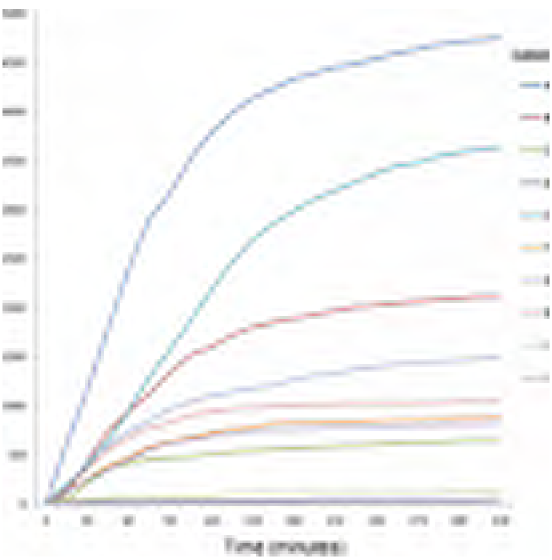

Figure 2 Changes with the lapse of time of accumulation of number of “tweet”

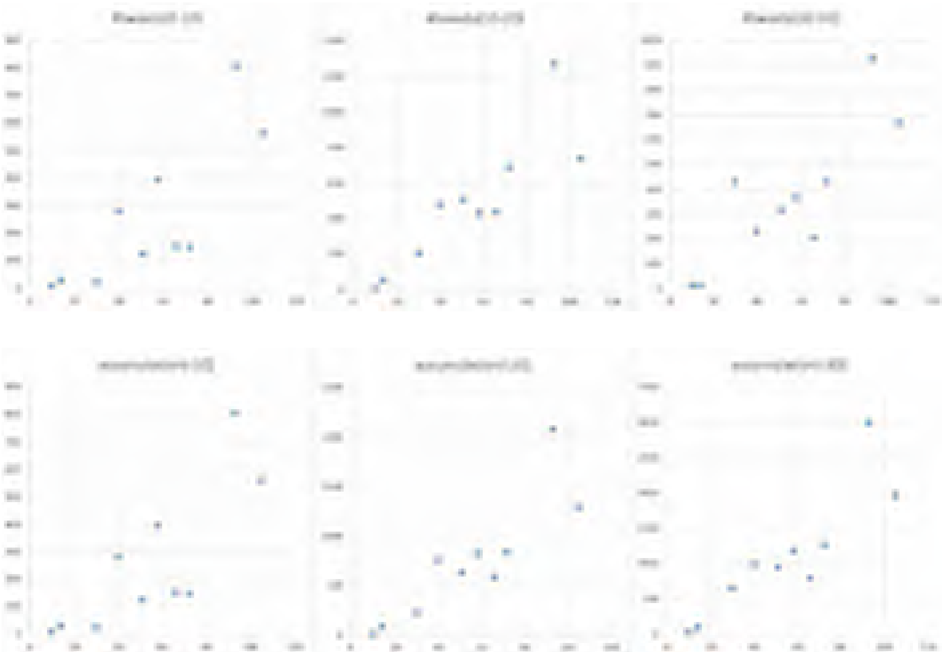

Figure 3 Relationship between number of tweet and suspension time

Figure 3 shows relationship between number of tweet and suspension time. Relationship between number of tweet and suspension time has positive correlation. As a result of this figure, regression analysis can be adopted in this study.

3 Suspension time forecasting model

3.1 Simple linear regression

Figure 4 shows that result of simple linear regression. Tweet data was delimited every five minutes and regression analysis was done. A red dot means observed data, and blue dot means estimated data. As seen in the figure, the reproducibility is high. When the twitter data until ten minutes is used, R is 0.8.

3.2 Exponential regression model

Figure 5 shows that result of exponential regression. Tweet data was delimited every five minutes and regression analysis was done. A red dot means observed data, and blue dot line means exponential regression. As seen in the figure, the reproducibility is high. When the twitter data until 5 minutes is used, R is 0.87 which is better than result of simple linear regression.

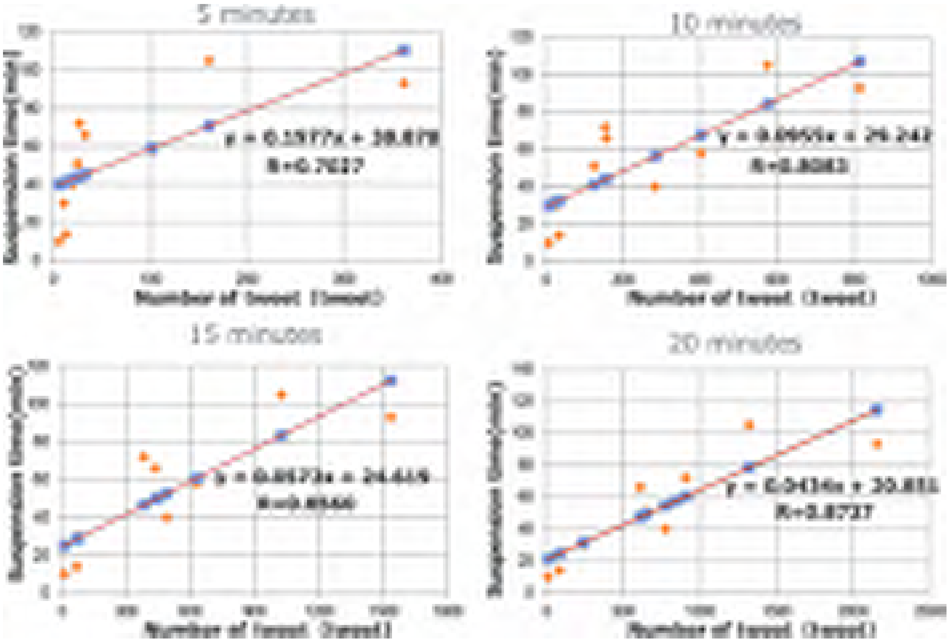

Figure 4 Simple linear regression

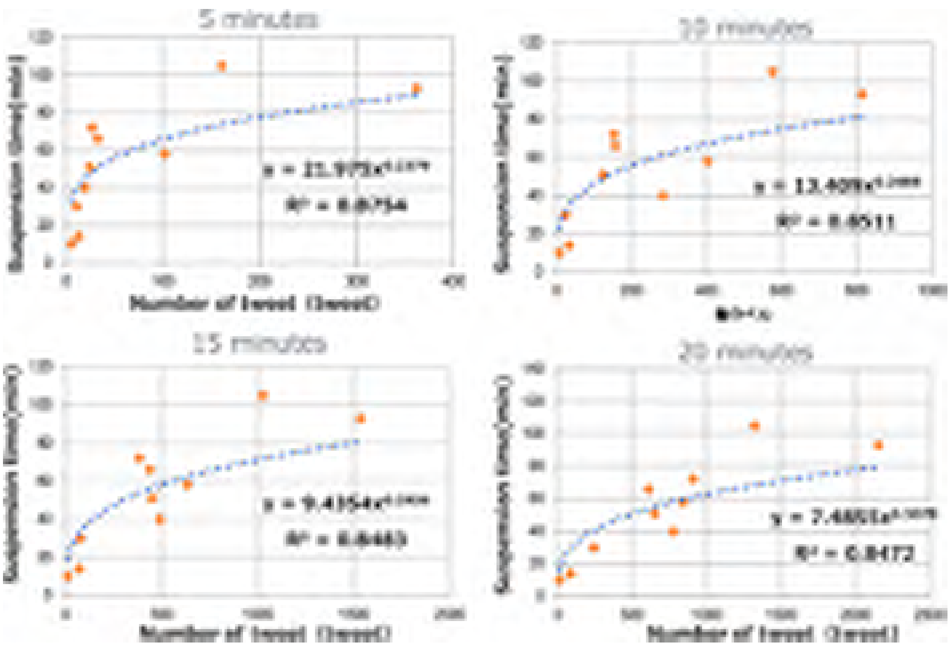

Figure 5 Exponential regression

4 Conclusions

In this study, number and contents of "tweet" are analysed under accidental conditions which are a small railway accident and a railway accident in which a lot of people get influenced. Furthermore, simple linear regression and exponential regression were applied to develop the suspension time forecasting model. Then, it became clear that in case of using simple linear regression, the reproducibility is high. When the twitter data until ten minutes is used, R is 0.8, and in case of using exponential regression, the reproducibility is very high than simple linear regression. When the twitter data until five minutes is used, R is 0.87 which is better than result of simple linear regression. As a result of this study, suspension time of railway accident can be forecasted using twitter data until 5 minutes by exponential regression model.

References

[1] Takada, K., Fujiu, M. & Yokoyama, S.: Perception of the Travel Time Reliability of the Urban Railway Service in Tokyo, Proceedings of the First Conference on Road and Rail Infrastructures – CETRA 2010, 17–18 May 2010, Opatija, Croatia, pp. 873-879.

[2] Takada, K., Fujiu, M.: Study of Willingness to Pay for Reducing Lost Time of Railway Users, Journal of Eastern Asia Society for Transportation Studies, Vol. 8, pp. 1464-1474, 2010.

13 PASSENGER SERVICES: BAGGAGE STORAGE AND BOARDING

STORE&GO+ – NEW PASSENGER SERVICES BY NEW BAGGAGE STORAGE ROBOTS

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Abstract

In order to gain attractiveness and high comfort at rail terminals the effective function and design of public baggage storage services gets an essential success factor in near future. At intermodal traffic stations the necessity of quick passenger transfers in short periods of time is very essential. In addition there will be increasing demands for real comfortable temporary deposits for luggage or shopping bags, too. Conventional luggage lockers are inefficient concerning utilization of space, which is limited at most railway stations. Effective depot services have to be accessible with ease for the passengers, too. Modern depot services also need to be available for old or handicapped passengers.
Store&Go+ is a new integrated concepts for modern railway stations. It provides multifunctional benefits for travellers, local service providers and the retailers at terminals. The new Store&Go+ depot service represents an innovative system for short term and long term storage of traveler's luggage at railway stations, but is also foreseen to be used as pick-up place for internet shopping or as parcel buffer at distribution hubs in the logistic industry.
Store&Go+ addresses the utilization of unused free space of buildings, increases safety and comfort for travelers. The System increases the attraction of public transport in general. Store&Go+ represents an innovative cube adaptive storage technology as well as a novel concept in effective warehousing and handling of parcels and luggage. It integrates a completely new boxing system and a warehousing robot. Store&Go+ can be used in house at railway stations and as outdoor module at open air platforms. This paper describes the experiences with a prototype and the learnings of user acceptance tests, executed by an Austrian research consortium.

Keywords: railway station, baggage storage, warehousing robot

1 Introduction

Future railway stations and similar hot spots of public transportation need to address changing mobility demands of travellers. In a research study funded by the Austrian "FFG" (Austrian Research Promotion Agency) requirements and demands for temporary storing possibilities of luggage in railway stations were investigated. This study [1] is based on a broad investigation of essential traveller requirements and expectations for future luggage storage systems. Modern public storage services should be available in short-term and medium-term simply to avoid troublesome handling of luggage and significant stress factors for all passengers.
The release from luggage and an easy to use deposit during waiting times at the station or for business or touristic reasons is an important factor to increase the attractiveness and efficiency of public transportation systems in general. Beside short travel connections and centrally located railway stations public transport also requires just comfortable depot systems.
After initial investigations and fact findings an Austrian research consortium developed a new and innovative depot system (named Store&Go+) as experimental prototype for passenger's

luggage or baggage. This Store&Go+ system represents a new cube adaptive technology for containers which can be top loaded at extremely user friendly drop stations. The system incorporates a novel concept for effective warehousing and handling of luggage, too.
The construction details provide integrated answer to the needs and expectations of all kind of potential users and address the needs and operational conditions of station operators, too. In order to integrate demands from different perspectives the Store&Go+ prototype was developed during a two-year inter-university research collaboration of the universities of applied sciences in Steyr, Wels and St.Pölten in alliance with the Viennese research company netwiss. The Austrian Federal Railways “ÖBB” and the leading Austrian logistics systems integrator “TGW” joined the research team as practical think tanks, too and the deve¬lopment was funded by the Austrian Research Promotion Agency “FFG”.

2 The new perspective for railway station operators

Public transport hubs (e.g. train stations or airports) of the new generation are changing more and more into multi-functional business centres and do not only offer the travel, but also a local shopping, business communication and shopping service. In this context, the troublesome handling of luggage constitutes a significant stress factor and a loss of comfort for all passengers.
The developed Store&Go+ concept provides an easy way of luggage deposit and overcomes the disadvantages of conventional locker systems (e.g. the difficulty of handling of heavy luggage, which is deemed acceptable to older or weaker travellers). Instead of side or front loaded lockers the Store&Go+ system uses specialised 3-sided containers which can be filled from the top and from the front side at automated loading stations in ergonomic working height.

Figure 1 Disadvantages of conventional locker systems compared with an ergonomic and easy to handle drop station

To protect the personal belongings of the user each container is covered and sealed automatically before its transfer into an automated storage area. Fully automated and industrially standardised storage and retrieval systems (“Commissioner”) – which are already well established and approved in many installations store the closed and locked container into a background racking system. In parallel the user gets a barcoded ticket to pick up his luggage again whenever he likes.
For the railway station (or airport) operators – who are very often in lack of commercial space – the partially patented Store&Go+ technology enables the efficient utilisation of free space heights within the station infrastructure as a storage room for passengers luggage containers. The Store&Go+ system represents a solution, both technically and economically explored and examined for feasibility, which can be used not only in the planning of new stations, but also for existing buildings. Even open air installations at very small stations or in pedestrian zones are technically possible.

The central idea of Store&Go+ is that travellers (or any other passers not traveling) can check in their baggage in close proximity to high frequented parts of the station – like near the front entrance, or the exit of subway stations, or at taxi stand – quickly and easily into an automated luggage depot. In this way, relieved from the efforts of luggage future passenger increasingly may make use of the commercial centre of the railway station (e.g. restaurants, shops, travel agents, communication infrastructure, catering or the facilities for local supply, etc.). Before departure or before leaving the station the customers also easily can retrieve their luggage again.
Through the development possibly existing acceptance barriers were countered in upfront, too. So the Store&Go+ concept is a technological answer to meet all the demands and challenges of modern depot services. The system is also foreseen to be used as pick-up place for internet shopping in a self-service automatic unit.

3 Ergonomics and the user's perspective

In order to enable best user acceptance and customer-friendliness a prototype of the drop and pick-up station was built before detailed final construction steps were executed. More than 150 persons in all categories were interviewed and had to fill out structured questionnaires after testing different possible ways of operation.

Figure 2 Store&Go+ prototype tests with all potential user groups

Beside the cost for the individual storage period an easy handling of luggage lockers is the essential criterion for user acceptance. The handling consists of easy locating and supported lifting up the luggage and of self-explaining software dialogues. Also fast return of the luggage is essential. Most important is also the size of the lockers and the fact whether the luggage must be lifted or not.
Depending on the age and sex travellers have got different difficulties when they must lift luggage. For example about 50% of all female passengers with large luggage are not able or willing to lift it, about 20% are able or willing to lift it up to about one meter and only 30% are able to lift it higher.
For about 70% of all female and 40% of all male travellers storing luggage at low level is important. Also for 70% of all passengers above the age of 60 this is a must.
The time needed for storing and especially for getting back the luggage is another very important criterion for acceptance. More than 25% of the asked train passengers say the luggage

returning must not need longer than one minute, more than 50% accept a time need between one and three minutes. The time need includes the whole process between coming to the locker until getting the luggage and leaving. Especially the subjectively felt time needed when passengers are in a hurry and they are nervous because of the approaching departure of their train is very important.
Many of today's lockers are too small for usual luggage items. The width of many lockers is 33cm but 40% of all luggage items are bigger than this size. That means 40% of luggage items do not fit into normal lockers. Passengers either cannot store it or must use a much more expensive locker for huge items.
It was analysed, that about 80% of passengers staying more than 30min at the station think about using a short term locker for easier moving in order to use the station infrastructure like shops or bistros. For half of them the handling must be very quick and cheap.

4 The Store&Go$^+$ system solution

With regard to an utmost universal use the Store&Go$^+$ system is designed for all the fields of railway transport, aviation and other hot spots of public places to be available for commuters, passengers and shoppers. But the focus of this development, however, must depend on railway stations.

4.1 The focus on passengers healthiness and comfort demands

Core of an automatic and public luggage storage system must be an ergonomic, robust and tailored design, which fits to the needs of all kind of user groups. Any differences, such as age, gender and physically or mental health should be irrelevant. This means that an old person shall be able to deposit luggage, such as anyone using a wheelchair or a woman with a baby buggy.
The user interface principally has to meet the requirement of the general population. The dimensional design for physiological user heights and gripping areas directly result from the ergonomic body reference. These requirements for a physio-friendly and ergonomic operation have to be supplemented with barrier-free equipment for people with technical restrictions (disabled).
Also the cognitive system requirement factors have to be taken into account for the software and screen design of the user interface. Multilingualism in the user dialogs and the use of readily understandable pictograms and colours have to be applied even for colour-blind people. All operating functions must include logical (= the common expectation appropriate) operating dialogues with any function cancel option for all starting and operating steps with online help function.
All these specific human related requirements have to be added by standardized safety and comfort requirements, ranging from rounded bearing edges for protection against bruises to the exclusive use of simples controls which avoid users to be jammed.

4.2 The technical solution

As part of the research project Store&Go$^+$ both the technical and economic feasibility has been proven. Finally the layout is characterized by the user friendly top and side loading philosophy and by a container volume adaptive handling technology.
The warehousing technology is based on proven and reliable automation concepts of industrial small part warehouses, which is combined with the innovative top and side loading container system to meet the conditions and demands as defined above. The key solution element is the cube adaptive container system which includes the covers (negative boxes) and cover locking.

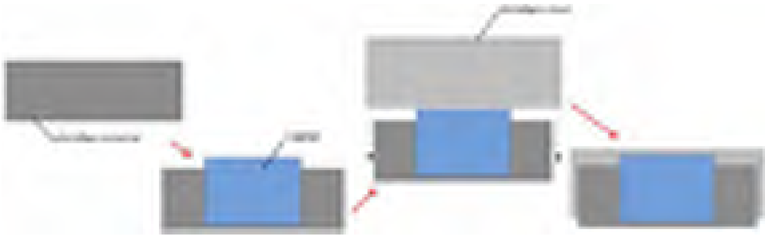

Figure 3 Store&Go+ container topping function

Requested stability and robustness soon results in a materials selection of metal and the following frameworks specifications:

- <u>Framework of the resulting criteria for the container construction:</u>
  - large dimension: length: 100cm, width: 60 cm
  - small dimension: length: 50cm, width: 60 cm
  - max. luggage height 50 cm
  - resistant against vandalism and stealing
  - generally easy and simple mechanic for low maintenance cost
- <u>Framework of the cover criteria:</u>
  - The cover has to be secured in both directions (lifting and lowering of the lid) in case of swelling luggage (e.g. when it opens autonomously)
  - Automatic capping and uncapping takes place inside of the system in order to avoid bruising of the passenger.
  - An automatic control of the filling level is necessary.
- <u>Resulting criteria framework for the construction of the Pick Station:</u>
  - Pick station have to operate for the luggage deposit and the pickup.
  - Because of performance reasons a pick station (luggage input and output terminal) shall not serve more than 150 stock locations (bins).
  - Payment shall be able to be settled by use of vouchers, international credit cards and money. Instead of returning cash back vouchers may be issued.
  - The output orientation of the container shall be the same as for the input task. This allows easier gripping the handles or straps of the luggage.
  - A control of the container filling levels is realised by use of cameras and scales.
  - Mounted mirrors support visible inspection of the luggage place¬ment in the container as well as the inspection of emptiness after outtake.
  - In case a container is not emptied completely by the customer, it must be stored again automatically.
  - The container must be transported underneath its supporting guides to the loading position in such a way, that no customer items are able to fall out. (e.g.: everything falling out of pockets has to fall into the container only.)
  - Any danger of bruising or crushing has to be excluded.
  - The handling height (minding disabled persons) is 40-60 cm.
  - The max weight of luggage shall be about 35 kg per container.
  - Initial height measurement or control of overhangs should be done before the move to the inner part of the station takes place. The control of emptiness or of maximal filling levels might be supported by pictures, which are taken within the station, where lighting conditions can be kept constant.

The automatic storage and retrieval system incorporates a lifting beam designed for optimum volume utilisation and maximum performance. The warehouse module may be installed across a length of up to 10m and can be used at building heights of up to 18 m. It accommodates a load handling device for single-deep storage of the covered containers.

Figure 4 Store&Go$^+$ commissioner as storage module

5 Conclusion

With the concept of Store&Go$^+$ a concrete technical system has been developed for the innovative deposit of luggage in self-control robots. It is characterized by both a volume adaptive container technology as well as a novel luggage storage technology using racking systems and conveyors. This directly addresses the quality objectives of an effective luggage deposit service, but also the performance targets and passengers comfort.
Travellers will purchase more at the station whenever they can deposit hindering items. Store&Go$^+$ enables station managers to meet this need and by means of offering a new attractive service they will increase the attractiveness of public transport in general, too. The easy to understand self-service function is not only a guarantee for good "usability" and user acceptance, but also facilitates urban mobility and stimulates the motivation to use public transport.
The Store&go$^+$ system addresses especially changing travel behaviours at rail-way stations, which increasingly will serve not only as enter or exit stations of public transport but also as centres for communication and the daily shopping. The replacement of existing locker systems by Store&Go$^+$ systems shall not only address the baggage depot service, but also has the potential for use dor the Lost &Found department at stations. Furthermore, the new system is ready as pick-up station of internet shopping and for the daily needs of commuters and other travellers. The internet and its constant accessibility via smartphones not only change the communication and mobility but also the purchasing behaviour of working people.
The vision of Store&Go$^+$ provides a functional response to the growing demands of the changing mobility behaviour at traffic stations. Travellers (or any other user groups who are not traveling) shall be able to quickly, easily and ergonomically place their luggage in automated self-service depots and always pick it up again quickly after a simple payment process.
The Store&Go$^+$ system is designed in modus to be implementable in adaptable sizes and scalable storage capacities at railway terminals or airports.

References

[1] Graf, H.-C., Rüger, B.: I2VPlus Endbericht für Kooperative Projekte und Konzepte, FFG-Projektbericht Store&Go, Wien 2011

[2] Graf, H.-C., Stadlmann, B., Rüger, B.: "Store&Go – The innovative system for automatic luggage storage services at railway stations", Proceedings EURO-Zel 2011 Challenges for European Railways, Tribun EU, ISBN 978-80-263-003-8, pp. 67-72, Jun. 2011

[3] Rüger, B., Graf, H.-C., Stadlmann, B.: "Luggage lockers – Needs and expectations of passengers", Proceedings EURO-Zel 2011 Challenges for European Railways, Tribun EU, ISBN 978-80-263-003-8, pp. 321-328, Jun. 2011

[4] Graf, H.-C., Rüger, B.: "Innovative, effective and customer friendly luggage storing", Proceedings Railway Terminal World Design and Technology Conference, Vienna, Austria, Apr. 2012

REQUIREMENTS ON FUTURE RAILWAY INTERIORS

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Abstract

Within the project FLEXICOACH in cooperation with Technische Universität Wien, Fachhochschule St. Pölten, Fachhochschule Joanneum, Siemens, netwiss and ÖBB Personenverkehr AG, passenger opinion surveys regarding their wants and needs were conducted. The aim of those surveys was to obtain information about everything a passenger requires; in order to get all information needed various subjects such as duration and frequency of journeys, activities during journey, well-being, stress factors, age, gender and group dimensions, were interrogated.

Keywords: railways, passenger requires, baggage, journey time

1 Introduction

Overall 3.826 questionnaires were analyzed. All questionnaires were conducted in summer 2012 on the Austrian Westbahn-line between Vienna and Linz. Due to the summer holidays a lower participation of students must be considered methodically. Furthermore and also due to summer vacations less rides to or from work are expected.
Approximately 50% of travellers undertake a trip lasting several days, around 10% are free time trips without an overnight-stay. Journeys in connection with education or work (rides from / to work, rides from / to education facilities, business trips either one-day or with a several day's duration) account for 25% of all journeys. The remaining 10% are to be allotted to private settlements.
Rail travellers mostly are young, approximately 12% are aged between 13 and 18 years, almost half of all interviewees are between 19 and 39 years old. 27% are part of the "40 to 60 years of age" group and around 12% are older than 60 years. The fact that children under the age of 12 are underrepresented is simply because they rarely fill in questionnaires.
54% of travellers are female, 46% are male. With the exception of people older than 60 years, in all age-groups female passengers form the majority.
Approximately one third of the passengers travel alone. Another third travels in a group of two persons. Around 11% travel in a group of three, 7% in a group of four and 12% in a group of five or more people. The journeys were classified in journeys up to 30 minutes, 30-60 minutes, 60-90 minutes, 90-120 minutes, 2-3 hours, 3-4 hours, 4-5 hours and more than 5 hours. Most journeys (respectively 20-30%) in all age-groups last between two and three hours. With increasing age the duration of journey as well increases slightly, short-term rides mostly are done by younger people. Summing up all general information gained, elderly people take the train less frequently, but if they do, they go for longer free time rides. In opposition, younger people take the train more often, and mostly for short business trips.
The better part of all interviewees quoted "comfort" as the major reason to take the train, around 40 % of all passengers declared "environment", "no car" and "price" as their major reasons to choose the train (see Fig. 1). "Safety" and "duration of journey" are an inferior aspect.

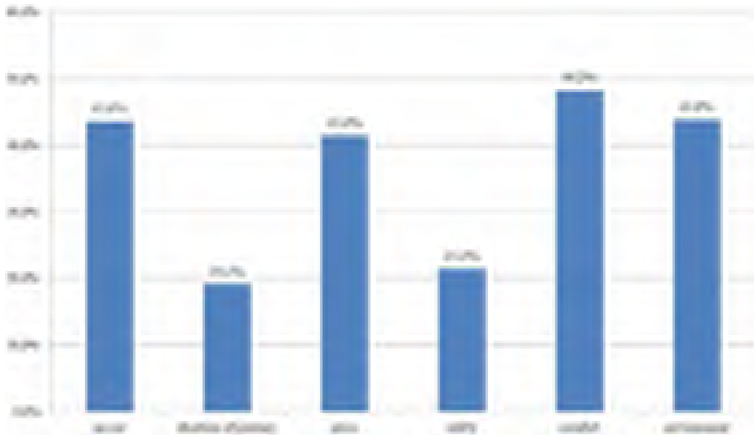

Figure 1 Reasons to take the train

2 Baggage

Regarding baggage, most information were attained from a diploma thesis [1], which treats issues of baggage transport on an extensive data basis. Amongst other things, various pieces of luggage were weight and measured. The accumulated x-, y- and z-dimensions of all luggage measured (not included is carry-on baggage) are demonstrated in Fig. 2.

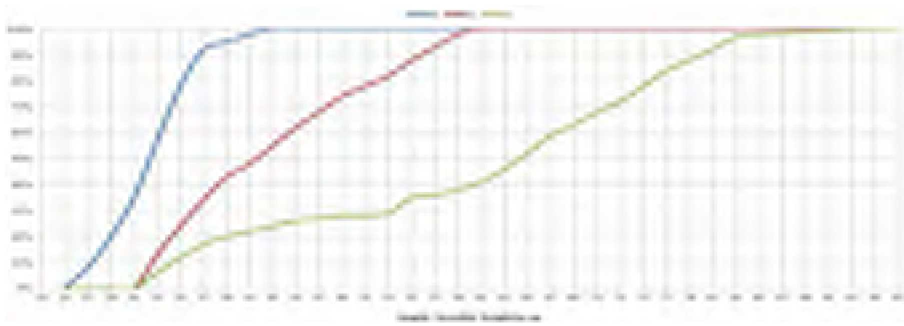

Figure 2 Dimensions of luggage

Those accumulated measurements can be used in order to design adequate storage between the seat backs or baggage racks. Analysis show that there are two main issues regarding baggage room. First passengers do not want to lift their luggage, and especially not to the height of overhead storage. This attitude is more common amongst women and increases with age. Second and due to security reasons, passengers wish to have their luggage in visual range. If these requirements are not met, passengers are very willing to store their luggage in not-intended place, like seats or corridors. This behaviour leads to a lower quality level and to a loss in capacity due to occupied seats.

3 Actual use of journey time

A major aspect was the purpose of the journey (business trip or free time ride). Every other business traveller declares to use his laptop, smart phone or tablet while travelling, while only one quarter of travellers on a free time ride uses those devices. Fig. 3 shows the details.

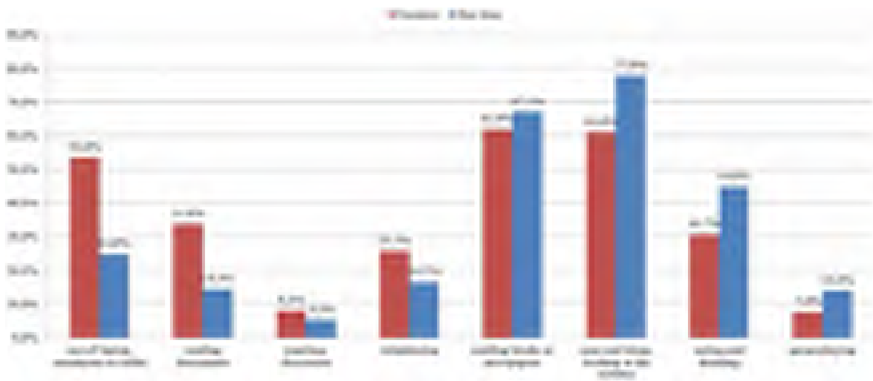

Figure 3 Actual use of journey time – influence of the purpose of the journey

Another major aspect regarding the use of the journey time is the age of the traveller. Generally speaking there is a slight decrease regarding actually performed activities with rising age. However activities need to be considers separately, while using electronic devices decreases with increasing age, activities like “looking at the scenery” or “reading a book / the newspaper” increase with increasing age.

4 Exercises on the train

The longer the journey the higher is need to move. Around 20% of passengers travelling up to an hour wish to exercise during their journey. The percentage rises to 40% when the duration of the journey rises up to five hours or more.

5 Desired use of journey time

Analysis (Fig. 4) shows that there is a connection between the use of journey time and the purpose of the journey. All too often (around 20% of all interviewees) passengers criticize missing mobile services. Because of several comments it is obvious that a missing WLAN-connection is intended.

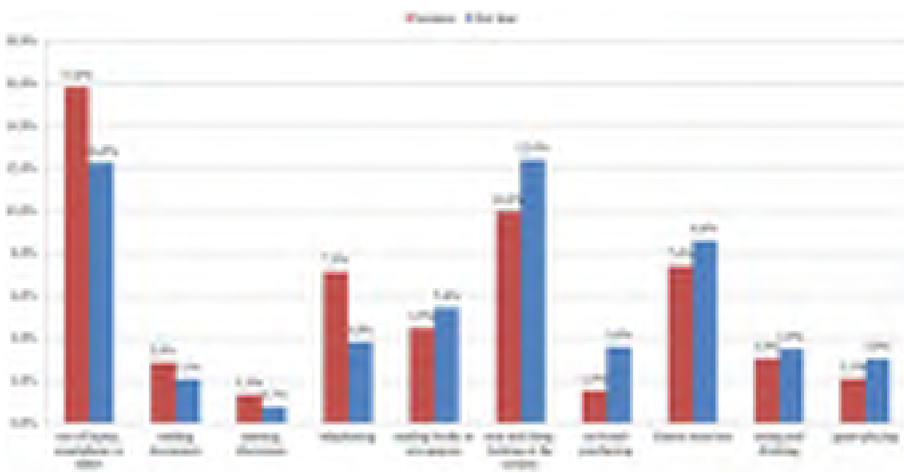

Figure 4 Desired use of journey time: influence of the purpose of the journey

Together with the absences of tables (respond 12% of all interviewees), this is the biggest obstacle when it comes to using tablet, smartphones and laptops. Around 17% of all interviewees criticize uncomfortable and fixed seats as well as absent silence, which holds them from their desired activity “ease and sleep”. The need to move reflects in the desire for a possibility to exercise. The age has significant influence regarding the desired use of travel time. The younger the interviewee, the more non accomplishable activities are quoted. Anterior passengers are significally more satisfied with the possibilities offered, respectively less frequently express a wish to use the time in another way. In Fig. 5, every desired activity is marked in a different colour (the lowest layer indicates “using the laptop”, the second lowest “using tablet or smartphone” and so on).

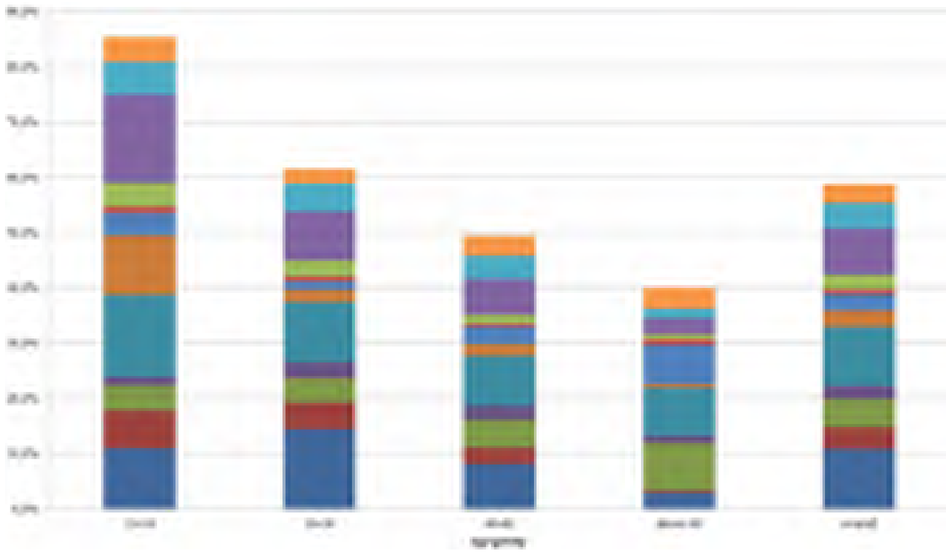

Figure 5 Desired use of journey time: factor age, cumulative percentage

Similar images with heavy age-related variation often occurred in the course of the examination, for instance regarding questions about well-being, stress factors, activities, etc. The journey time is a major aspect when it comes to desirable use of time. The longer the journey time, the more requirements were quoted, in particular if the duration excesses two hours.

6 Well-being

Around 33% of the interviewees feel “very well” when travelling by train, about 52% feel “rather good”, 14 % feel “rather bad” and only 1 % of the passengers do not feel well during train journeys. The assumption that those outcomes correspond with the fact that younger passengers mostly are on business trips, while anterior passengers use the train prevailing for free time trips, is unfounded. Around 50% of all train journeys are leisure time trips, lasting several days. Interviewees between 13-18 years, the group that feels most uncomfortable during train journeys, mostly goes on free time rides without an overnight stay. Thus there is no obvious connection between the purpose of the journey and the well-being.
Passengers travelling first class are feeling better than passengers travelling second class. Furthermore there is a strong connection between the well-being of the passengers and the degree of capacity. Therefore on weekdays from Monday to Thursday passengers mostly feel “rather well” or “very well”, while travellers on Fridays and weekend feel “rather bad” or “not

well”. The higher degree of capacity during weekends leads to an oftener nomination of stress factors like “high degree of capacity”, “search for seat”, “noise” and “fellow passengers”.

7 Stress factors

The stress factor most frequently nominated was “search for a seat”, around 20% of the passengers feel stressed (see Fig. 6). Also sensed as stress factors were “high degree of capacity”, “noise” and “fellow passengers”. The factors most frequently mentioned are those which appear at a high degree of capacity and obviously lead to a deterioration of the well-being.

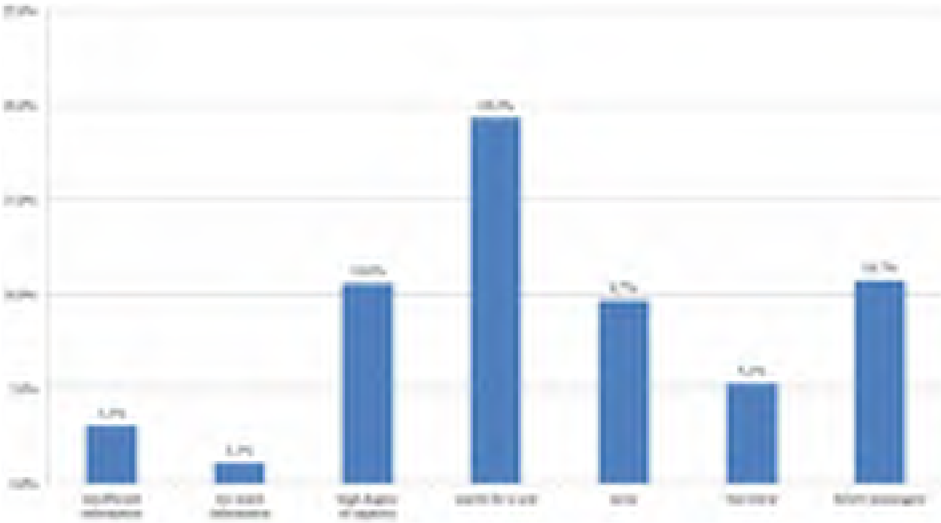

Figure 6 Stress factors during the current journey

Analogue to the well-being, the age of the passengers is crucial when it comes to cognition of stress. Younger passenger are more stressed than anterior passengers, at least they quote it more often. With increasing age (groups from 13-18 years, till 60 years) the nomination of arising stress factors during an actual train journey decreases under 50%. Considering the most frequently quoted stress factor “search for seat”, the nominations in the age group “over 60 years” decrease even to a third of those from passengers between 13-18 years. This is a very notable fact, because precisely this stress factor was assumed to arise with increasing age, also in terms of luggage. Generally speaking, anterior passengers are more satisfied with the frame conditions than younger ones.

8 Service features

Most frequently nominated when it comes to service features were “reasonable priced meals” (31%), “purchase of newspapers” (24%), “transmission of knowledge” (23%), “entertainment” (20%), “possibilities to exercise “ (18%), “relaxation practises” (17%). With increasing age the interest in service features heavily decreases.

With increasing journey duration, the interest in service features increases as well. Passengers on a free time ride show more interest in service features than travellers on a business trip. Around two-thirds of all respondents show interest in healthy nutrition during their train journey. This desire is more common under female passengers than under male ones.

9 Atmospheric environment

The outcomes of this particular subject won't be discussed any further. Generally speaking, the nominations made by the passengers are very subjective and do not always refer to any comprehensible objective criteria. For instance, in every sort of train the temperature was between 25 and 26 °C, nevertheless there are different sensations and evaluations regarding the temperature, which can be connected to the sort of train. The highest percentage of satisfied passengers is to be found in trains of the private operator Westbahn (over 80% satisfaction). The average registered temperature is exactly identical to the temperature registered in the Railjet-trains of ÖBB. However, Railjet was evaluated ten percentage points less than Westbahn. It is obvious that not only the temperature, but rather the general well-being or the consciousness of a deliberately taken decision to travel with a new operator (Westbahn war operating only eight months at the moment of the opinion survey) contributed to this outcome.
The opposite way around also Railjet was not only evaluated regarding temperature, but rather general well-being (for that matter Railjet scores rather low). There is a similar effect notable when it comes to train categories (first or second class). Passengers travelling first class rated the atmospheric environment higher than passengers travelling second class, there was no objective difference however.
It has to be considered that subjective sensations had a great influence also on questions regarding well-being, illumination and stress factors. Advanced studies would be very helpful in order to interpret the outcomes in the right way.

10 Conclusion

Compared to other modes of transportation the railway system has got the big advantage that passengers are able to use the travel time efficiently. This is one of the biggest advantages of competition. Unfortunately today's vehicles hardly offer the requested equipment which allows the best possible time use. Train passengers are very interested in using the travel time for working, mostly on technical devices like one note books or tablets, for reading or for relaxing. For efficient time use the investigations have shown that the individualisation of the space in the train is essential. It is important that all passengers can follow their requested activities without affecting other passengers. For example people who are working may produce noise but on the other hand need calm for concentration. Additionally they need light whereas people who want to sleep need it dark and calm. So further investigation must focus on how the space in the vehicle can be individualized in a best possible way.

References

[1] Plank, V.: Dimensionierung von Gepäckanlagen in Reisezügen, Wien 2008

[2] Esveld, C.: Modern Railway Track, Second edition, TU-Delft, 2001.

[3] Plank, V., Rüger, B.: "Grundlagen für eine optimierte Gepäckunterbringung in Reisezügen"; ETR – Eisenbahntechnische Rundschau (eingeladen), 07+08 2009 (2009), 07+08; S. 417 – 421.

[4] FLEXICOACH, Endbericht AP2, Nutzerbedürfnisse, Wien 2013

PUBTRANS4ALL – ACCESSIBLE BOARDING INTO OLDER COACHES

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Abstract

Regarding to EU regulations today's public transportation systems must be accessible for everyone without any restrictions. The relevant question is: How can trains be accessible for everyone? The huge variety of different vehicles and different platforms does not allow level boarding everywhere, only in so called "closed" systems. The paper gives an overview about the requirements for new boarding assistance systems and about the decision making process referring to a new developed lift system for UIC-coaches. This lift system is developed in the EU-founded project PubTrans4All.

Keywords: trains, older coaches, boarding assistance system

1 Introduction

The result of the previous work in the PubTrans4All-project, founded by the EU, led to the decision that the most important step towards an accessible rail system at the moment is the development of a boarding assistance system (BAS) for existing UIC wagons. These cars are still in use in large number all over Europe. Due to design limitations it is not possible to retrofit these types of vehicles in order to use existing BAS. So at the moment only platform based BAS can be used for wheel chair users. For all other types of vehicles some kind of BAS exists (lifts for high speed trains, ramps for low floor trains). The aim of further research in this project was to develop a BAS that can be used for installation in UIC wagons.
The layout of older UIC coaches and modern high speed trains that are designed for wheelchair users and other PRMs in general is similar. UIC coaches has small doors with a width of 800, while in modern trains the door width is increased to 900 mm. The difference is that there are already lift solutions for a door width of 900 mm but none for narrower doors. The UIC coach has doors located at the end of the coaches. Because of the folding or sliding steps as vicinity of the buffers as well as other constraints, there is no space under the steps for the installation of a BAS. Additionally, the space at the coach end is occupied by mechanisms of the head doors leading to the next coach, fire fighting equipment, some electrical components etc. Typical for these coaches is that the passageway is in majority cases at one side outside the longitudinal centre line of the vehicle because of the neighbouring toilet cabins adapted for people with handicaps and persons with reduced mobility. Finally, there are usually only two potential positions left which could be used for stowing the BAS.

2 General requirements for a new boarding assistance system

The general requirements provide an overview of all relevant parameters that must be considered when designing a new boarding assistance system. Table 1 presents the importance scores used in order to rank the evaluation criteria. Table 2 summarises the requirements. Features rated as not important, are not shown herein.

Table 1 Criteria importance scoring

Score	Meaning
1	Very important – critical to successful operation (“must have”)
2	Important – high benefit for users and operators (“nice to have”)
3	Less important – some benefit for users and operators, but not absolutely necessary

Table 2 BAS evaluation criteria – overview

User with devices	wheelchair, walking frame, baby prams	1-2
Physical impaired	Walking disabled, with crutch or sticks, elderly, diminutive people	2
User with special needs	Visual and hearing impaired	2-3
General passengers	Passengers with luggage, children, pregnant	2-3
Operation without staff	Operation by passengers themselves, automation	2
Operator		
Reliability of BAS	Prevention of Malfunction	1
Operational quality	Short dwell time, malfunctions must not influence train operations	1-2
Operational effort	Number of staff	1-2
Failure management	Problems easy to solve	1
Manufacturing/ Implementation		
Universalism	The system needs to be universal, retro-fitting allowed	1-2
Costs	Costs as low as possible	1
Manufacturing effort	The manufacturing effort needs to be low – especially when retro-fitting	1-2
Safety		
Safety risks	No safety risks to be tolerated	1
Safety features	Optical and audio signals	1-2
Maintenance		
Maintenance effort	Number of personnel required, special tool required	1
Costs		2
Sustainability	recyclability and energy consumption	3
Aesthetics		
Optical design	Aesthetics is important for customer acceptance	2-3
All regulations must be fulfilled (currently according to TSI-PRM) as a minimum standard. Some specifications in project PT4All have been set higher than required.		

3 Decision making process

At the beginning of the project the consortium consciously set the bar very high in order to get the best possible results. The primary defined goal of the project was to find a technical solution to provide accessibility to all passengers in all boarding situations. To get innovative and completely new ideas, a student competition was also initiated. The consortium believed that students don’t have the detailed knowledge about railway vehicles and they are therefore more independent in their thoughts. Experts usually have a tunnel vision because they think too much about reasons why something cannot work.
After a long research and discussion process including the excellent ideas from the competition, the consortium concluded that many restrictions are necessary and the all-in-one

solution is not possible. At this point it must not be forgotten that the PubTrans4All project is a research project which also has the goal of demonstrating what is and is not possible.
In the first step, current and future plans of the different railway systems over the whole of Europe have been analyzed in order to identify the biggest gaps.
For all local systems (including busses, tramways, metros, urban and suburban railway traffic) a newly developed BAS is neither necessary nor meaningful. All these systems can be seen as so called “closed systems”. Here the operators provide vehicles which correspond to the existing platform height; which means level boarding is provided. If level boarding is not yet provided, then operators plan to adapt the platforms and/or their vehicles. Local traffic operators in general don’t want to use technical devices (BAS) because of operational time reasons. Level boarding is in general the best solution for travelers and for operators. It is the only situation which really offers accessibility to all passengers. Furthermore, the passenger flow in the station can be speeded up which means a shorter dwell time and therefore advantages for operators.
To offer level boarding it is necessary that the platform and the vehicle floor have a common height and the remaining horizontal gap between vehicle and platform is bridged. For that many technical solutions already exist. For all situations where level boarding is not possible, different approved technical solutions such as ramps or lifts already exist.
Compared to the local traffic systems; high speed, long distance and international railway traffic will not be able to offer level boarding for the following two reasons: The first reason is that because of static, high speed trains need a higher floor. The lowest floor height in high speed trains is offered in Talgo-trains (760 mm). All other vehicles have got higher floor height. The second reason is that in the TSI two different platform heights are defined as European standard (550 mm and 760 mm). That also means for the next decades all international trains will need to stop at both levels!
Furthermore, the investigation has also shown that actually within the next decades a huge number of high floor vehicles will run in European countries in long distance traffic. Due to the long life cycle of railway vehicles they can’t be changed in a short or medium term. So the decision was to develop a BAS for all types of high floor vehicles. In general there are four possibilities – ramps or lifts, platform or vehicle based.
The operators’ surveys clearly show that operators either plan to provide level boarding in the future or – everywhere they cannot – they strongly wish to have vehicle based systems. Two reasons can be identified for that wish: Firstly, operators want to be independent from the infrastructure and want to offer the possibility of accessible boarding everywhere. Secondly, it is very difficult to provide a platform based device at all (!) platforms in a railway network.
In order to provide accessibility to all passengers, ramps seem to be the only possibility because lifts cause a big bottle neck if every passenger trys to use one door. But here the big problem is that it was not possible to find a technical solution for installing a ramp system into existing vehicles. Furthermore, ramps must be very long if they will be used for high floor vehicles.
Because of the impossibility of finding any technical solution for ramps in existing high floor vehicles, the decision was to focus on lift systems for existing high floor vehicles. For the next steps of development two decisions have been necessary: Who the user will be and which vehicles are relevant.
The investigations show that for all types of high floor trains with an entrance door width of at least 90cm, different lift systems already exist. It is not meaningful to develop another system because passenger and operator surveys have shown that the existing systems work well enough. But there is one very big group of high floor railway vehicles in Europe, the so called UIC-wagons. This is a unique type of vehicle which will be running in many European countries for some decades more. In many countries the UIC-wagons form the backbone of the long distance railway traffic, especially in eastern European countries. But due to many construction limitations described in previous deliverables no technical solution has yet been

developed. Therefore, the consortium came to the decision that the most important step to offer accessibility to all is to focus on UIC-coaches!
A lift system under very limited frame condition means many restrictions and compromises. In regard to user requirements, wheelchair users are the only passengers for whom a technical solution is an absolute must. For many other groups it would be very nice to have some technical devices; but if there is no chance, than other solutions are acceptable. As other solutions, special services at the entrance door are recommended within this project. There already exist good examples in different European countries which can be advanced. At the end of the decision process, it came out that the most important case is to develop a vehicle based BAS for UIC-coaches. Since there are many restrictions because of the vehicle design, it has also for this situation been necessary to define some "compromise solutions" regarding the construction. All recommendations for a vehicle based BAS for UIC-coaches are shown in the next chapter "Detailed technical requirements for a BAS for UIC wagons".

Figure 1 Decission making process

4 Technical requirements for a BAS for UIC wagons

As described in the chapter "decision making process" the consortium decided to focus on a BAS that can be implemented into UIC wagons, Table 3. Therefore, at this point all technical requirements that have been identified especially for the implementation into UIC wagons will be described in detail.

Table 3 Applicability of a BAS in different vehicles

Characteristic	Value	Comment
Carrying capacity	300 kg	Covers 99% of wheelchair users
Minimum clear width of lift platform	720 mm	Covers 96% of wheelchair users
Minimum platform length	1200 mm	
Maximum working height difference vehicle floor-platform	1300 mm	
Distance from the side of the coach when the lift platform is in lowered position:	as small as possible, but not less than 75 mm	The lowest foldable stair required to be lifted up before descending of the lift platform.
Boarding/alighting parallel to the vehicle	recommended	Alternatively, exit sideways through lay down of the side fenders (required for narrow platforms)
Handrail bound to the platform on one side, should be at the height of	650 to 1100 mm from platform level	
Integrated folding seat for categories of users other than wheelchair users	Recommended	
Finger pressure for activation of control buttons	≤ 5 N	
Manual force to operate the lift by staff	≤ 200 N	For example for emergency mechanical activation.
Manual force to operate the lift by staff at movement start	≤ 250 N	Allowed only for short period at the start. For example for emergency mechanical activation.
Vertical speed in the operation	≤ 0.15 m/s	Movement should be smooth
Operating speed variation: empty-maximum loaded	±10 %	
Speed of any point of BAS without load	≤ 0.2 m/s	Up to 0,6m/s is allowed by EN 1756-2. To meet TSI PRM, maximum speed without load no more than 0,3m/s is recommended.
Acceleration during operation with load in any direction and at any point of the lift platform	≤0.3 g	
Tilting speed of the lift platform	≤ 40 m/s	In case of automatic adaptation to the relative angle between vehicle and platform, for example at superelevated track by platforms in curves.
Automatic roll-off protection height	≥100 mm	The barrier in front and at rear side of the wheelchair lift platform should be automatically erected during lift operation.
Lateral side guards height:	≥25 mm min ≥50 mm preferred	Prevention of the wheelchair side roll-off from the lift platform
End of travel mechanical limitation devices	yes	
Prevention of any unauthorized operation in the absence of the operator	yes	Locking and unlocking by a key or a code or similar.
Overload protection of the main power electrical circuit		Fuse, an overload cut-out or similar
In stowed position BAS must be safe against uncontrolled displacements. Mechanical securing devices dimensioning according to the accelerations:	alongitudinal: 5 g alateral: 1.5 g avertical: 1 g	These accelerations can arise in the exceptional case of occasionally buffing impact at coach staying in yard (without passenger) (UIC 566)
Activation possible only at:	V = 0 km/h	

Table 3 Applicability of a BAS in different vehicles (continued)

Activation of the BAS should introduce activation of the coach brake system.	yes	Movement of the train during BAS usage must be prevented
Minimum safety coefficient against yield strength	2.1	
The lift platform surface should be smooth and must have slip-resistant surface	yes	Slip resistance according to EN ISO 14122-2.
Easy removal of ice and snow must be possible	yes	
Gaps or holes in the platform area shall not accept a probe greater than:	15 mm diameter	
Illumination of the lift working zone	yes	
The warning devices should be fitted at edges that can come in contact with persons or injure passengers or personal.	yes	light / reflective stripes / reflective markings, visible at night also
Visual and audible warning signals during the lift movement must be activated	yes	
The operation control should be of type hold-to-run.	yes	Lift shall stop moving and remain motionless after the control is released.
Movement no more than 100mm for any part of the lift platform after release of the control is tolerable to slow lift down	yes	Mechanical drives with self-braking capability or with independent direct acting brakes, or hydraulic systems with normally closed valves etc. should be used.
Controls shall be designed to avoid unintentional lift actions.	yes	Recessed or covered buttons, two hand controls, etc.
One control position is recommended	yes	Conflicts of commands must be avoided
In any case of breakdown, it is acceptable that platform may decrease with controlled speed:	≤ 0,165 m / s	For example in hose or pipe failure by hydraulic systems or similar.
Safety devices shall preferably operate through active positive action.	yes	
A stop in overload protection should be present at overload more than	25%	
An emergency stop button within reach of the user should be present	yes	Release of the emergency stop button should only be possible by the personnel
Additional protecting measures such as obstacle detector, foot entrapment protection etc.	recommended	Although control of hold-to-run principle is used additional measures are recommended
During lift platform closing the risks of crushing or shearing of the arms or head must be avoided.	yes	Limitation of the closing force, security cut-off, etc.
Other technical details not covered in this table preferably should be based on:	TSI PRM, EN 1756-2, RVAR	

5 Outlook – Conclusions

Providing accessible rail transport to all passengers is nowadays a must. This is because of different national and European regulations but also because of ethical questions. That means every person must be able to use a public means of transportation. In light of this, the entrance to railway vehicles and the whole boarding process is a big challenge and causes huge difficulties.

In order to be able to provide accessible boarding to all passengers, the consortium tried to define the biggest gaps that must be closed. For mid and long term thinking the results can be summarized as follows: Because level boarding is in the process of being or will be offered soon for all types of local, urban and suburban traffic; no systems are required. At this point, only horizontal gaps need to be bridged. Therefore, enough technical solutions already exist. In the rare case that level boarding is not possible, existing technical solutions can be used. For all high floor vehicles with an entrance door width of at least 90cm, enough technical solutions such as different lifts exist. A new development is neither meaningful nor necessary. The intensive investigations of the consortium led to the result that for the huge number of UIC-wagons which are running and will be running within the next decades all over Europe no vehicle based BAS yet exists. There are too many design limitations.
Due to the fact that UIC-wagons will still form the backbone in many European railway networks within the next decades; it is absolutely necessary to develop a BAS for this operation.
Due to the different limitations resulting from the vehicle construction, it is also necessary to make several compromises. But the developed compromise allows about 99% of all actual wheel chair users to board a UIC-coach. In combination with a good personnel service at the entrance, which is also recommended in this project, the UIC wagons can also become accessible for nearly all passengers.

References

[1] Rüger, B., Tauschitz, P., Petutschnig, B.: Boarding Assistance System Evaluation Criteria Report, deliverable 2.1, June 2010. EU-FP7-Project Public Transportation – Accessibility for all.

[2] Rüger, B., Tauschitz, P., Petutschnig, B.: Existing Boarding Assistance System Evaluation Matrix Report, deliverable 2.2, August 2010. EU-FP7-Project Public Transportation – Accessibility for all.

[3] Simic, G., Rüger, B., Petutschnig, B., Tauschitz, P., Milkovic, D.: “Recommendations for Improving Boarding Assistance Systems”; December 2010. EU-FP7-Project Public Transportation – Accessibility for all.

[4] Rüger, B., Tauschitz, P.: D 4.4 – Vehicle based BAS prototype design and evaluation April 2012. EU-FP7-Project Public Transportation – Accessibility for all.

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