UNIVERSITY OF ŽILINA

TRANSCOM 2011

9-th EUROPEAN CONFERENCE
OF YOUNG RESEARCH AND SCIENTIFIC WORKERS

PROCEEDINGS

SECTION 7

CIVIL ENGINEERING

ŽILINA June 27 - 29, 2011
SLOVAK REPUBLIC

UNIVERSITY OF ŽILINA

TRANSCOM 2011

9-th EUROPEAN CONFERENCE
OF YOUNG RESEARCH AND SCIENTIFIC WORKERS

under the auspices of

Ing. Eugen Jurzyca
Minister of Education, Science, Research and Sport of the Slovak Republic

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SECTION 7

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Edited by Jozef Jošt, Peter Brída

ISBN: 978-80-554-0376-2

TRANSCOM 2011
9-th European conference of young research and scientific workers

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Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

The Use of Fine-Grained Waste Mineral Materials in the Base Course in the Recycling Technology with Foamed Bitumen

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Abstract. The fine-grained waste mineral materials is used in the recycling technology with foamy bitumen as mineral filler from 5% to 20% in the aggregate mix. This possibility has allowed for using.in design mix recycling., fine-grained waste mineral materials from the dedusting system of aggregate in coating plant and during the production of aggregates in mines (whilst crushing the rocks and rinsing the aggregate)..

In research used three different fine-grained waste mineral materials (dolomite, gabbro, and quartzite sandstone). It was defined basic functional properties ei. pH – acidity- alkalinity, Blaine's actual area Pw , the contents of clay minerals – with the use of methylene blue indicator MBF and determination of the voids of dry compacted filler. The fine-grained waste mineral materials are proportion in amounts 10%, 15% and 20% to mineral mix. On the basic of the analysis which was fall within the domain of stability and flow (Marshall), indirect tensile stretch ITS and water resistance TSR, it was defined of dependency of functional properties of fine-grained waste mineral materials on the mechanical basecourse properties in recycling technology with foamy bitumen.

The possibility of utilization of fine-grained waste mineral materials in the recycling technology of deep cold with foamed bitumen, substantially affect the protection of the environment while behavior the required properties of recycled pavement.

Keywords: Fine-grained waste mineral materials, recycling, basecoure, recycled mineral mix, foamed bitumen.

1. Introduction

The fine-grained waste mineral materials incomes during the process of producing mineral mix asphalt, i.e. during the aggregate dedusting process and during the production of aggregates in mines (whilst crushing the rocks and rinsing the aggregate).

Strict environmental regulations have been imposed to determine the amount of industrial dust emitted to the atmosphere as $100 mg/m^3$. The above-mentioned regulations require the introduction of an alternative dedusting system, i.e. bag filters. Virtually all amount of fine-grained waste mineral materials will be kept. Thus obtained a fine-grained waste mineral materials are difficult for recycling and it is virtually impossible to use this material again.

The preliminary analysis allow use the fine-grained waste mineral materials in the recycling technology of deep cold with foamed bitumen and this technology it is possible to try utilization a fine-grained waste mineral materials. The mineral mix in the recycling technology with foamed bitumen may contain from 5% to 20% dust less than 0.075 mm.

2. The Research of the Properties of Fine-Grained Waste Mineral Materials

The purpose of research in the aspect of the application of fine-grained mineral material in the deep cold recycling with foamed asphalt, were three types of mineral dust, which differed each other a place of origin and mineralogical composition. Two of them are received from the dust collection system of the asphalt batch-plant such as: dolomite (D) and Gabbro (G). The fine-grained material consisted of sandstone quartzite (K) is obtained as a washing process on mine. The next step of research was a defining functional properties, such as: specific surface by means of Blaine tester (Pw), the contents of clay minerals - MBF ratio, an indication of voids fraction conctent of dry and compacted filler AVR.

The study of basic functional properties is to determine the utility of mineral dust in the aspect of using for a base coarse with foamed bitumen in deep cold recycling with foamed asphalt. The results of the basis of functional properties were presented in Table 1 including the determination of the coefficient of variation for the considered parameters.

Type of fine-grained waste mineral materials	TYPE OF TEST	ARITHMETIC MEAN	V [%]
G (gabbro)	P_W	4709	0,4
	MB_F	3,3	8,2
	pH	7,5	4,7
	AV_R	56,84	0,4
K (quartzite)	P_W	3487	0,6
	MB_F	5,0	5,4
	pH	4,5	4,1
	AV_R	56,88	0,5
D (dolomite)	P_W	4209	0,8
	MB_F	1,3	10,9
	pH	7,5	2,4
	AV_R	55,36	1,3

Tab. 1. The researched types of fine-grained waste mineral materials.

The maximum value of the Blaine's specific surface for fine-grained waste mineral materials were received from gabbro (G), and dolomite dust (D). The lowest finding has been observed for fine-grained waste mineral materials obtained from the quartzite (Q) sandstone.

The largest value of methylene blue indicator MBF, which is content of clay minerals in the studied dust, characterized a waste material from a quartzite sandstone which could be caused by the content of shale.

The most acidic material is quartzite sandstone with a pH is less than 4.5. The fine-grained mineral material has a pH value the same as rocks from which it was obtained. It should be noted that all the tested dusts reached a similar value of AVR [%] on level between 55 and 57%. The largest value of AVR [3] reached the dust from the sandstone quartzite, while the smallest the dolomite aggregates. This may be related to the hardness of dust, especially derivation of received grain dust.

3. Design of Mineral Mix

In order to assess effect of the addition of fine-grained mineral material to the mineral mix with foamed bitumen in deep cold recycling were designed three type of mineral mix which differ the content of mineral dust content differing (20%, 15%, 10%) and its origin.

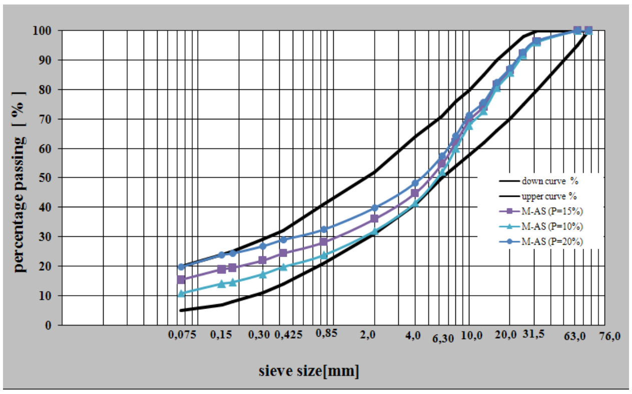

Fig. 1. The area of proper granulation of mineral mix with foamed bitumen for base course [8].

The mineral mix with foamed bitumen [Fig.1] consisted of local recycled asphalt pavement, dolomite aggregate 31.5 mm which was earlier a compound of mechanical base course treatment and mineral dusts used for increasing a fraction content (gabbro, quartzite sandstone and dolomite). As the foamed bitumen was used a Nyfoam85 in amount of 4.0% [4]. To ensure a more stiffness of pavement, the cement additive was used in amount of 1.5%.

4. Design of Mineral Mix with Foamed Bitumen

In order to assess the impact of fine-grained mineral material on the mechanical properties of the base course deep cold recycling with foamed asphalt it was designed a research program consisting of two stages.

In the first stage it was defined the basic physical and mechanical properties of recycled base course such as: stability, stiffness in accordance with Marshall methodology and indirect tensile strength [5].

The second stage was referred to the study on determination of resistance to the water effects by using of TSR ratio [2].The results are presented in graphically form in Figure 2 and 3, while the second stage in Figure 4 and 5

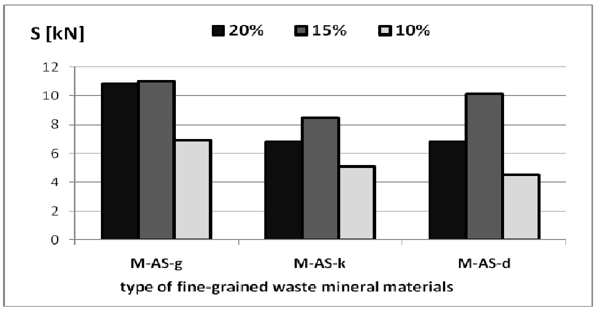

Fig. 2. The stability according to Marshall.

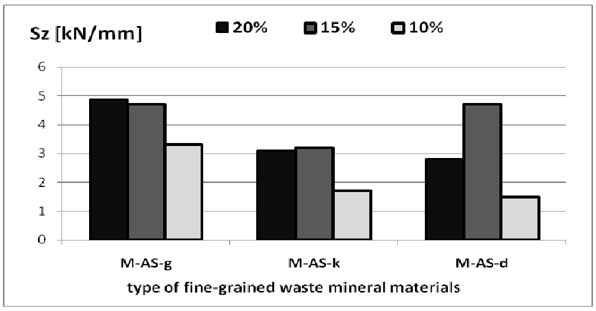

Fig. 3. The stiffness according to Marshall methodology.

The analysis of test results of sub base in recycling technology, It was found that that the base course witch foamed bitumen and gabbro fine-grained mineral material was characterized by the highest values of stability according to Marshall in all range of content of mineral dust (M-AS) [2], the rest mixtures with the same concentration of the waste material was characterized by a much smaller value of the stability [Fig. 2].

The value of stiffness according to Marshall is a function of stability. The greatest value of the stiffness [Fig.3] were reached all mixtures of M-AS-g in the scope of dosage of fine-grained mineral material - 20%, 15% and 10%.

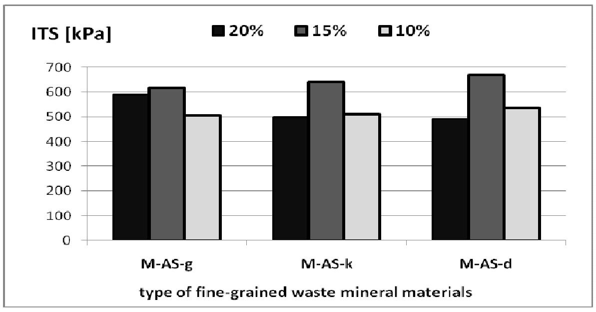

Fig.4. Indirect tensile strength (ITS) base course.

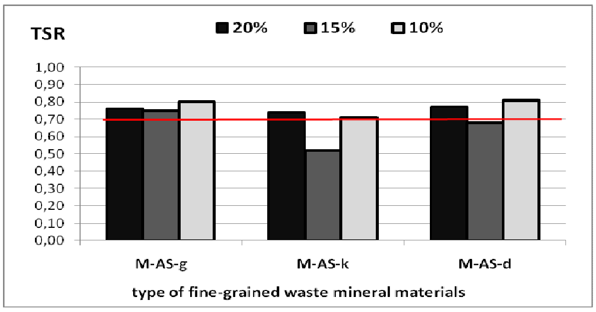

Fig. 5. Water sensitivity ratio (TSR) base course.

The technology of deep cold recycling mixes with foamed bitumen, with dolomite and gabbro fine-grained mineral material is resistant to the effects of water in all variant of mineral dust [Fig. 4]. The minimum value of the ITSR ratio [Fig. 5] was obtained in all mixes which waste material was incorporated in amount of 15%. It should be noted that the recycled base course, are resistant to the effects of water according to the criterion TSRmin = 0.7 [2]. Only the mix marked M-AS-k at a concentration of mineral dust of 15% is vulnerable to the effect of the water equal 0,58.

5. Conclusion

Based on the analysis of the test results of base course in the recycling technology with foamed bitumen following conclusions can be drawn:

- in order to apply the fine-grained mineral material is necessary to identify its characteristics and especially determining the value of pH, content of clay, the specific surface and volume of voids,
- the best values of stability and stiffness according to Marshall methodology for all the recycled mixes were obtained at a concentration of 15% fine-grained waste material,
- all mineral mixtures with foamed bitumen and waste material are resistant to the effects of water, at a concentration of 20% of grains less than 0.075 mm,
- due to the functional properties of mixes of in the recycling technology, the most beneficial is using a waste material from the gabbro aggregate in amount of 15%.

References

[1] GRZYBOWSKI A., 8/1997. Drogownictwo *Stosowanie do betonów asfaltowych mączki mineralnej z układu odpylania otaczarki* : 241-244.

[2] WIRTGEN COLD RECYCLING MANUAL. Wirtgen GmbH, Windhagen, Germany 2006.

[3] GRZYBOWSKI A., 1/2001. Drogownictwo. *Kryteria oceny przydatności wypełniaczy do mieszanek mineralno-asfaltowych*. str. 25-28.

[4] IWAŃSKI M., CHOMICZ A., 8/2006 Drogownictwo. *Przydatność stosowanych w Polsce asfaltów do spieniania,* s. 267-271.

[5] RAFALSKI L., *Podbudowy drogowe*. IBDiM. Warszawa 2007r. str. 110 - 116

Assessing the Effect of Iron Ions Adsorbed on Activated Carbon and the Efficiency of Decomposition of Organic Impurities Using Selected Oxidizing Agents

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Abstract. This study analyzes the effect of Fe(II) ions adsorbed on activated carbon on the efficiency of phenol decomposition, using H_2O_2, Fe^{2+}/H_2O_2 (Fenton's reaction) and 1:1 HNO_3 solutions (in the third case, in the presence of microwaves), and, accordingly, their effect on the sorptive capacity of regenerated activated carbons. The chemical regeneration of the activated carbonsusing the AOP method resulted in partial oxidation of the adsorbed organic substance. The presence of Fe(II) ions improved the efficiency of the regeneration process. The oxidation of the adsorbed phenol, however, caused a significant loss of mass of the activated carbon.

Keywords: Activated carbon, sorption, regeneration, oxidation, heavy metals.

1. Introduction

Removing toxic organic substances from wastewater is a difficult and complex problem and can be achieved by applying physical, chemical, electrochemical and biological methods. Wastewater from the chemical, petrochemical, coke and pharmaceutical industries is particularly difficult to treat, as it contains aromatic organic compounds such as phenol and phenol derivatives. Today, the Advanced Oxidation Process (AOP), which involves using H_2O_2 or Fenton's reaction (Fe^{2+}/H_2O_2), is becoming a more and more popular method of treating organic industrial waste [4,5,9]. In this case, hydroxyl radicals generated in the reaction environment are the oxidizing agent [4-9]. An alternative is applying the process of oxidation of the organic compounds previously adsorbed on activated carbon [4-9]. The literature on the subject, for example Refs. [4,6,8 and 9], confirms that activated carbon, added to hydrogen peroxide or Fe^{2+}/H_2O_2 solutions and organic substances, not only plays the role of a sorbent but also catalyzes the process of formation of hydroxyl radicals. Since the formation of hydroxyl radicals during a Fenton's reaction is directly related to the presence of Fe^{2+} ions, it is essential to determine whether the presence of this metal on the surface of activated carbon affects the effectiveness of the oxidation of the adsorbed organic substances. In this study, the problem was analyzed using the example of oxidation of phenol adsorbed on activated carbon.

2. Experiment

* *Isotherms of sorption of Fe(II) ions and phenol from aqueous solutions on virgin and regenerated activated carbons*

0.5g samples of different activated carbons were weighed into conical flasks and treated with 200cm^3 of solutions containing Fe(II) ions with concentrations ranging from 1.5 mg/dm^3 to 40 mg/dm^3 and phenol (Ph) with concentrations ranging from 50 mg/dm^3 to 750 mg/dm^3. The flasks were then shaken for three hours. The sorption isotherms were determined for phenol, Fe^{2+} ions, and phenol and Fe^{2+} ions in the following order: F-300-Ph, F-300-Fe-Ph, F-300-Ph-Fe and F-300-(Fe/Ph).

** Determination of the concentrations of Fe^{2+} ions and phenol*
The metal concentration was determined through MERCK Spectroquant tests using a Spectroquant NOVA 60 spectrophotometer. The concentration of phenol was established using a gas chromatograph equipped with a Thermo Scientific MS Focus GC detector and a TRACE-TR-1MS column, operating at temperatures of 40-260^{o}C.
**Regeneration of activated carbons by oxidation of the adsorbed organic impurities:*
a. using hydrogen peroxide
Five-gram samples of different activated carbons, saturated with phenol and iron ions, were weighed into flasks and treated with 200 cm^3 of a hydrogen peroxide solution with a concentration of 0.7 M/L and a pH of 8.1. The samples were then mixed for one hour. The hydrogen peroxide solution was decanted, and the activated carbons were rinsed first with acidified distilled water (1 x 100 cm^3), then with distilled water (4 x 100 cm^3), and, finally, dried at a temperature of 378^{o}K.
b. using Fenton's reaction
Five-gram samples of the analyzed activated carbons, saturated with phenol and iron ions, were placed in 200 cm^3 conical flasks and treated with 100 cm^3 of distilled water. Then, $FeSO_4$ and H_2O_2 solutions (with a pH of 3-4) were added simultaneously, maintaining a weight ratio of 1 : 5 between Fe^{2+} and H_2O_2. The samples were then shaken for three hours. Subsequently, the solution with Fenton's reagent was decanted, and the carbon was rinsed first with a basic solution (1x30cm^3) with a pH of ~ 8 to stop the oxidation reaction and then with distilled water (4x100cm^3) and, finally, dried at a temperature of 378 K until a solid mass formed.
c. *using an HNO_3 solution in the presence of microwaves*
The carbon samples were treated with a 1:1 HNO_3 solution (the proportion being 1g of regenerated activated carbon per 10 cm^3 of the oxidizing solution) and then heated for 5 minutes using a microwave field with a frequency of 2450 MHz produced by a Plazmatronika UniClever microwave mineralizer at 80 % of the generator's power. The solution above the carbon was decanted three times. The regenerated activated carbons were rinsed with distilled water until a neutral reaction was reached. Then, they were dried at a temperature of 378 K until a solid mass formed.
Characteristics of the activated carbons
The porous structure was determined by conducting low-temperature adsorption of nitrogen (77 K). The isotherms of adsorption and desorption were established with the volumetric method by means of a Sorptomatic 1900 analyzer. The total content of the acidic groups was determined through a titration analysis with a 0.01M HCl solution by titrating the excess of the unreacted 0.01M NaOH, which was previously used for treating the carbon samples.

3. Discussion of Results

In this study, we analyzed the sorption of phenol (Ph) on virgin F-300 activated carbon (F-300-Ph) and F-300 activated carbon with previously adsorbed Fe(II) ions (F-300-Fe-Ph), the sorption of Fe(II) ions on the carbon with previously adsorbed phenol (F-300-Ph-Fe) as well as the simultaneous sorption of phenol and iron ions (F-300-(Fe/Ph)). The activated carbon selected for the tests, F-300, is characterized by an extended porous structure (Table 1) and moderate surface acidity. The sorptive capacities of virgin F-300carbon in relation to phenol and Fe(II) ions were 180 mg/g and 5.5 mg/g, respectively (Table 2). It was also found that the previously adsorbed Fe(II) ions did not reduce the sorption of phenol. The results indicate that iron ions were sorbed on active centres other than phenol, but they actively participated in the process. The findings correspond to the results presented in Refs. [1-3], which suggest that the sorption of metals takes place on acidic oxygen groups on the surface of the activated carbon, while the presence of basic functional groups contributes to the sorption of phenol. Moreover, the presence of metal improves the sorption of phenol as a result of donor-acceptor interactions. The sorption of the metal ions on the activated carbon with adsorbed phenol was lower, i.e. 5 mg/g for Fe(II). This indicates that, because of their size, the adsorbed particles of phenol cover the active centres capable of sorption of the metal ions or change the chemical character of the surface of the activated carbon. The sorption from the metal ion-phenol mixture, F-300-(Fe/Ph), is much lower; it is 4 mg/g and 110 mg/g for Fe(II) and phenol, respectively. This might be a result of the competition in the diffusion region as well as the occurrence of donor-acceptor interactions between phenol and iron ions in the solution.

Samples	Oxidation conditions	S [m^2/g]	V [cm^3/g]	Surface acidity [mmol/g]	Mass loss [%]
F-300	-	965	0.57	0.57	-
F-300/HNO_3	activated carbon/1:1 HNO_3/ 2450MHz/80%/5min	850	0.72	2.2	25
F-300/ H_2O_2	activated carbon/ 0.7 M H_2O_2/L /60 min	960	0.68	1.6	12
F-300/Fe/ H_2O_2	Fe : H_2O_2 1:5/ 60min	910	0.70	1.8	22

Tab. 1. Characteristics of the virgin F-300 and regenerated carbons.

F-300 activated carbon-component sorbed as the first–component sorbed as the second (sorption from the mixture)	Sorptive capacity for iron ions, mg/g	Sorptive capacity for phenol, mg/g
F-300	-	180
F-300-Fe-Ph	5.5	170
F-300-Ph-Fe	5.0	180
F-300-(Fe/Ph)	6.0	110

Tab. 2. Comparison of the sorptive capacity of the virgin activated carbon, F-300, in relation to iron ions and phenol from aqueous solutions, according to the order of sorbed substances.

Saturated with phenol and iron ions, the samples of activated carbons were then subjected to chemical regeneration using the following oxidizing agents: a 1:1 HNO_3 solution, in the presence of microwaves [3], a 0.7 M H_2O_2 solution with a pH of 8.1, which, as stated in Ref. [4], contributes to the formation of OH^* radicals, and an Fe^{2+}/H_2O_2 solution with a pH of 3-4 (Fenton's reaction), responsible for the formation of OH^* radicals. It should be noted that the amount of oxidant in the reaction environment was selected in such a way as to ensure oxidation conditions according to stoichiometry.

First, it was vital to assess the influence of the oxidants on the F-300 activated carbon. The results in Table 1 show that the oxidants were responsible for the changes in the porous structure as well as the chemical properties of the sorbent surface. The greatest changes were due to the action of 1:1 nitric acid (V) on the activated carbon. At the boiling point of the solution exposed to microwaves, there was a substantial decrease in the surface area from 965 m^2/g to 850 m^2/g, and a simultaneous increase in the pore volume, which suggests destruction of the micropores and a rise in the volume of mesopores. The high acidity of the surface testifies to a considerable increase in acidic functional groups. The presence of hydrogen peroxide as well as Fenton's reagent results in a significant increase in the surface acidity after oxidation, but the changes in the porous structure are negligible. It should be emphasized that in each case there was a considerable loss of mass of the activated carbon.

Applying analogous conditions of oxidation for carbons saturated with phenol and metal ions, i.e. Fe(II), resulted in partial oxidation of phenol. It was possible to reuse the activated carbons to remove the substance from an aqueous solution. The results in Table 3 indicate that:

- the partial oxidation of phenol, occurring after treating activated carbon with an H_2O_2 or an Fe^{2+}/H_2O_2 solution, was not dependent on the amount of adsorbed Fe(II) ions or the order of sorption; the partial oxidation allowed resorption of 100 mg/g of phenol; the sorption, however, was higher in the presence of Fe(II) ions adsorbed by the activated carbons; direct action of hydrogen peroxide did not lead to phenol oxidation;

- applying a 1:1 HNO_3 solution on carbons saturated with phenol and Fe(II) ions caused a considerable decrease in the sorptive capacity of the carbons in relation to phenol from 180 mg/g to 90 mg/g due to oxidation; the sorptive capacity of the carbons with regard to iron ions was higher for carbon saturated with phenol only (F-300-Ph) than for virgin carbon (F-300).

In studying the efficiency of oxidation of phenol adsorbed on the F-300 activated carbon, it was found that the oxidation was only partial under the predetermined conditions, maintaining the stoichiometric ratio of the adsorbed organic substance to the oxidant. This might be due to the fact that phenols adsorbed on activated carbons are prone to polymerization, which is likely to change the oxidation process. Moreover, the amount of oxidant required for the reaction may be higher than when the process takes place in a solution. Another reason for the partial oxidation of phenol might be the reaction of the oxidant with the carbon matrix, which results in a considerable loss of mass of the activated carbon. The presence of metal on the surface of the activated carbon is also responsible for a greater loss of the sorbent mass.

Regenerated activated carbon	Regenerating/oxidizing agent	Sorptive capacity of regenerated activated carbon, mg/g			
		Unary solutions		Mixture (Fe/Ph)	
		Fe	Ph	Fe	Ph
F-300	H_2O_2	4	120	4	120
	Fe/ H_2O_2	6	110	6	100
	1:1 HNO_3	6	80	6	90
F-300-Fe-Ph	H_2O_2	2	110	2	120
	Fe/ H_2O_2	3	100	3	100
	1:1 HNO_3	3	90	3	90
F-300-Ph-Fe	H_2O_2	2	120	2	120
	Fe/ H_2O_2	2	110	2	100
	1:1 HNO_3	3	90	3	90
F-300-(Fe/Ph)	H_2O_2	2	120	2	120
	Fe/ H_2O_2	2	100	2	100
	1:1 HNO_3	3	90	3	90

Tab. 3. Assessing the sorptive capacity of the spent F-300 activated carbon according to the regeneration conditions and the order of sorbed substances.

The results show that the chemical regeneration of the activated carbons saturated with phenol using such oxidants as H_2O_2 and Fe^{2+}/ H_2O_2 resulted in partial oxidation of the adsorbed organic substance. The presence of adsorbed Fe(II) ions increased the efficiency of the regeneration, and, in consequence, the sorptive capacities of the regenerated activated carbons. However, the oxidation of the adsorbed organic substance led to a considerable mass loss of the activated carbon.

The work reported herein has been undertaken as part of project N N205 1993 33 funded by the Ministry of Science and Higher Education. In the academic year 2010/2011, Anna Picheta-Oleś, who is a PhD student at the Kielce University of Technology, has been awarded a grant co-financed by the EU European Social Fund within the Programme for the Development of the Educational Potential of the Kielce University of Technology: Education for success, Agreement No UDA-POKL.04.01.01-00-175/08-02, Priority IV, Measure 4.1, Action 4.1.1.

References

[1] BANSAL R. Ch., GOYAL M., *Activated Carbon Adsorption,* 2005, CRC Press.

[2] BINIAK S., PAKUŁA M., SZYMAŃSKI S., ŚWIĄTKOWSKI A., *Effect of activated carbon surface oxygen- and/or nitrogen containing groups on adsorption of copper (II) ions from aqueous solution*, Langmuir, 1999, 15, 6117-6122.

[3] DĄBEK L., *Regeneracja zużytych węgli aktywnych (Regeneration of spent activated carbons)*, Monografie, Studia, Rozprawy, Wydawnictwo Politechniki Świętokrzyskiej, M1, Kielce 2007.

[4] HUANG H.H., LU M.C., CHEN J.N., LEE C.T., *Catalytic decomposition of hydrogen peroxide and 4-chlorophenol in the presence of modified activated carbons,* Chemosphere, 2003, 51 935-943.

[5] LUIS A., LOMBRAÑA J.I., VARONA F., MENÉNDEZ A., *Kinetic study and hydrogen peroxide consumption of phenolic compounds oxidation by Fenton's reagent,* Korean J. Chem. Eng., 2009, 26(1), 48-56.

[6] RICHARD S., HORNG R.S., TSENG I.-CHIN, *Regeneration of granular activated carbon saturated with acetone and isopropyl alcohol via a recirculation process under H_2O_2/UV oxidation*, Journal of Hazardous Materials, 2008, 154, 366–372.

[7] TEKIN H., BILKAY O., ATABERK S.S., BALTA T.H.,CERIBASI H., SANIN D., Filiz B. DILEK F.B., YETIS U., *Use of Fenton oxidation to improve the biodegradability of a pharmaceutical wastewater,* Journal of Hazardous Materials, 2006, B136, 258–265.

[8] TOLEDO L.C., SILVA A.C.B., AUGUSTI R., LAGO R.M., *Application of Fenton`s reagent to regenerate activated carbon saturated with organochloro compounds,* Chemosphere, 2003, 50, 1049-1054.

[9] VIDIC R.D., SUIDAN M.T., SORIAL G.A., BRENNER R.C., *Effect of molecular oxygen on adsorptive capacity and extraction efficiency of granular activated carbon for three ortho-substituted phenols,* Journal of Hazardous Materials, 1994, Vol. 38, 373-388.

Elemental Analysis of Sediments from the Urban Storm Water Drainage Depending on Granulometric Fractions

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Abstract. This is an investigation of the physical and chemical properties of sediments from the urban storm water drainage to determine the relationship between its quality and particle size. The catchment area of 85 ha is located in the center of the city with the main thoroughfares. The study included: granulometric analysis, content of mineral and organic substances, elemental composition and Scanning Electron Microscope (SEM) analysis . The results showed presence of eleven particular fractions, content of mineral substances on the level 80,46 %. and organic substances on the level 19,54 %. Article presents also the results of studies of elemental composition and SEM analysis in the sediments depending on the factions to confirm significant part of minerals.

Keywords: Runoff sediments, sewers, granulometric analysis, Scanning Electron Microscope, elemental composition.

1. Introduction

One of the most important elements for the proper functioning of the city is an efficient system of urban storm water, which is responsible for proper collection and treatment of rainwater also called rainwater. Rainwater consists of water and sediments.

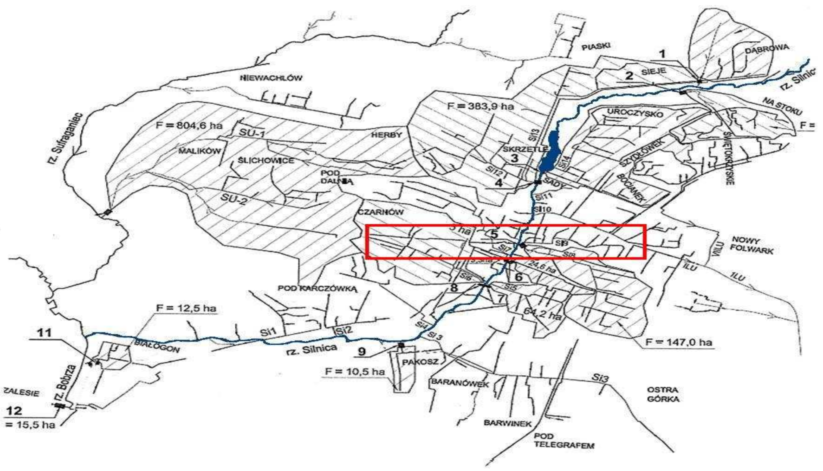

Fig. 1. Location of the catchment selected for research.

Figure 1 shows the location of catchment area selected for research. The catchment area of 85 ha is located in the center of the city with the main thoroughfares [1].

2. Materials and Methodology

The sediments was taken from the bottom of the settling chamber using special sampler. Settler is located in the underground part of the basin with the separator of oil products. Materials were taken from the entire cross-section of diameter 3 m settling chamber, so that in a way representative gave a deferred nature of the sediment. This means that the sediment samples of 1 kg from three different points of the bottom of the chamber and the results were averaged arithmetically [2].

a. Granulometric analysis

Granulometric analysis of sediment was carried out using a combination of sieving and areometric analysis according to the methodology contained in publication of E. Myślinska [3]. Hydrometric analysis was performed using a set of Hydrometer Eijkelkamp.

b. Content of mineral and organic substances

The content of mineral and organic substances determined by burning a weighed sludge previously dried to constant weight at 823 K. The residue after burning were weighed and identified as minerals, while the amount of organic matter determined by the difference of the masses and a predetermined amount of mineral substances.

c. Elemental composition

Elemental analysis was made using X-ray microanalysis by EDAX GENESIS APEX 4.

d. Scanning Electron Microscope analysis

SEM analysis was made using Microscope QUANTA FEG 250.

3. Results

In the first stage of the research carried out particular granulometric analysis to determine particles size distribution of sediments from urban storm water system.

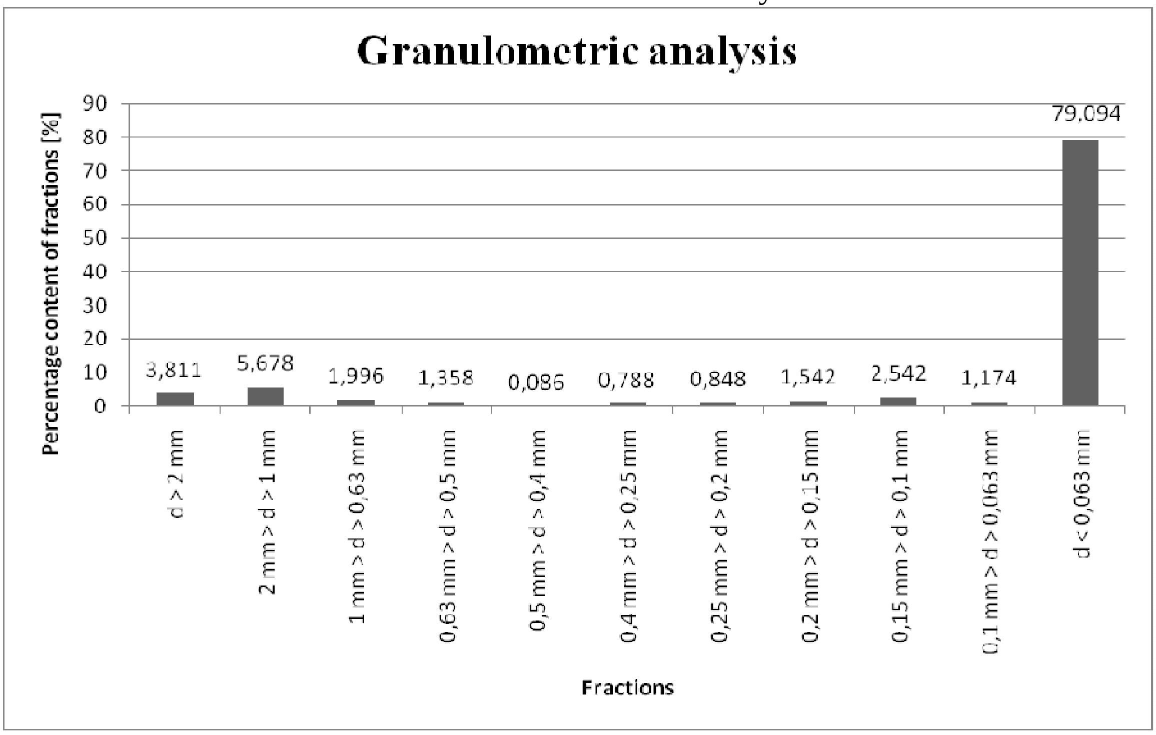

Fig. 2. Particular granulometric analysis of sediments.

Figure 2 presented the results of the particular granulometric analysis and shows the advantage of the sieve fraction of dust grains with a diameter of less than 0.063 mm which represents 80% of the tested sediment. The fraction of particles with diameters greater than 2 mm fraction defined as gravel is less than 4% of the test sediment and largely consists of organic matter (leaves, insects). Fraction with a diameter of 2 mm> d> 0.063 mm is a sand fractions which contribution in the settlement was set at 17%. Sieve analysis was complemented by an analysis which identified hydrometric by the smallest fraction of the size distribution. Results of analysis showed no particles with a diameter of less than 0.002 mm, which indicates that the slightest fraction of the analyzed sediment must be considered in its entirety as dusty fraction.

Granulometric analysis was complemented by Scanning Electron Microscope observations with elemental microanalysis.

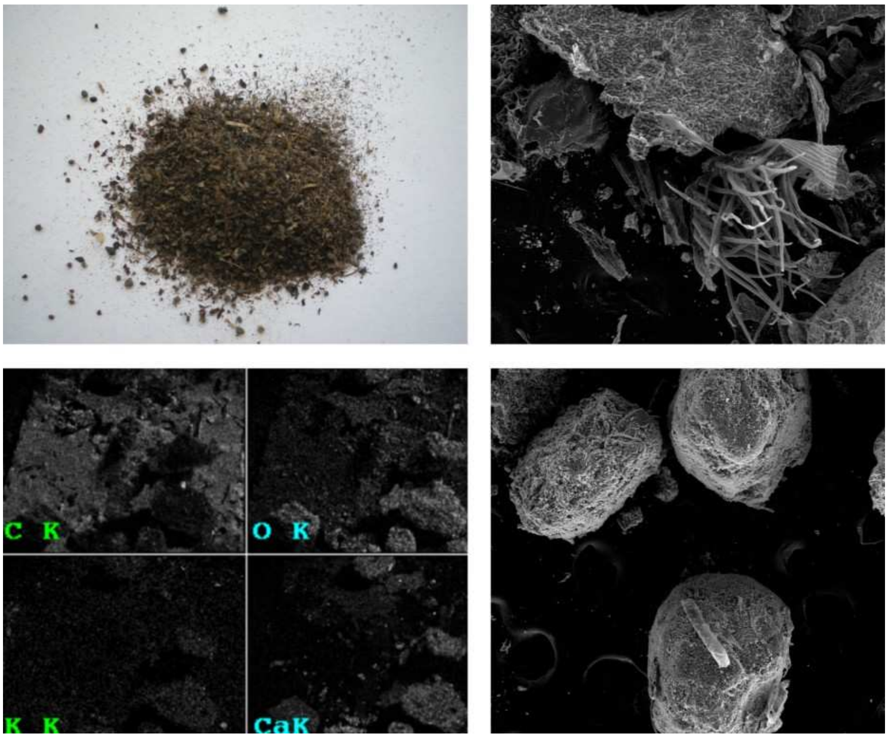

Fig. 3. Gravel fraction of sediment analyzed by SEM x 240 and elemental X-ray microanalysis.

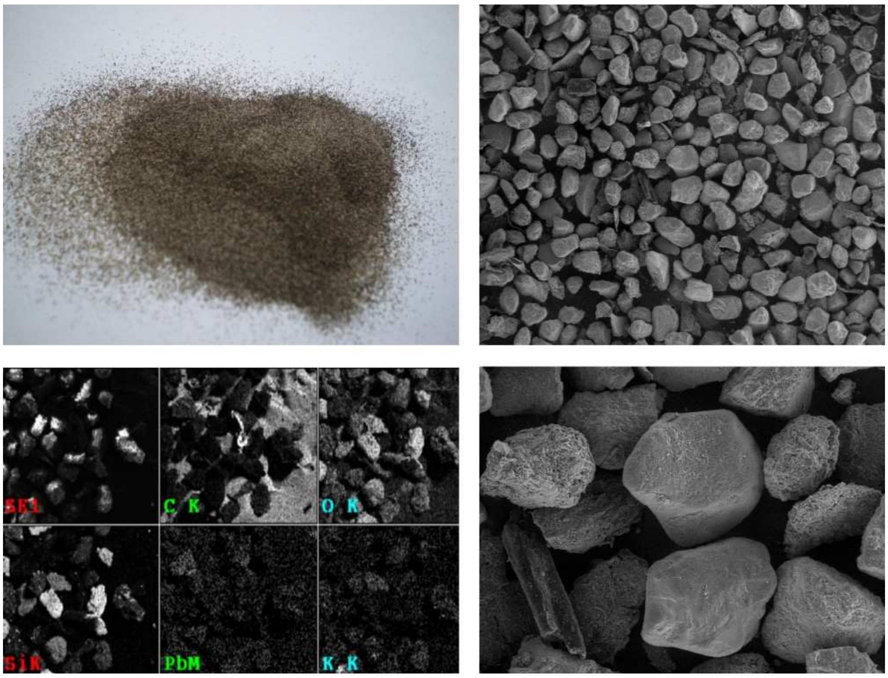

Fig. 4. Sand fraction of sediment analyzed by SEM x 240 and elemental X-ray microanalysis.

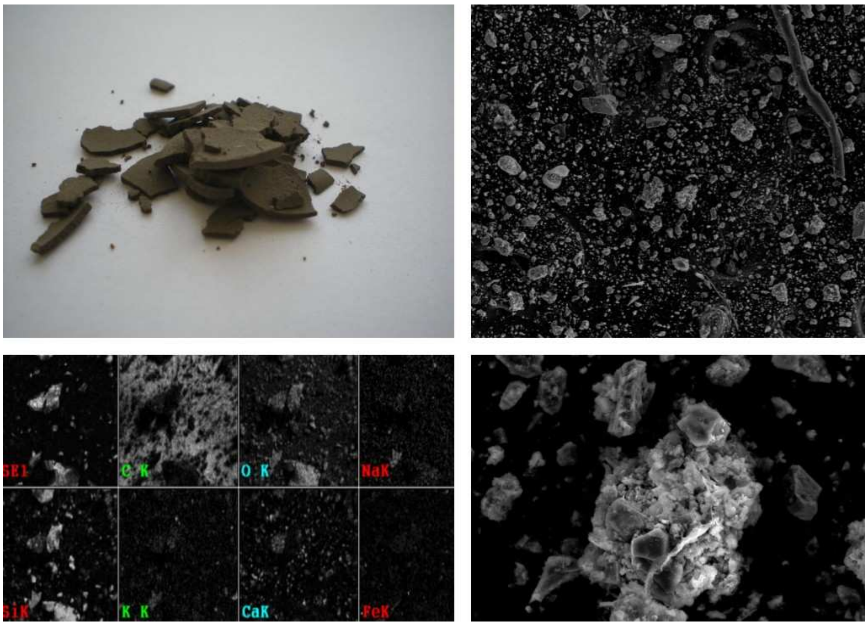

Fig. 5. Dust fraction of sediment analyzed by SEM x240/x2000 and elemental X-ray microanalysis.

Figures 3-5 presents SEM observations with elemental analysis for tree major fractions: gravel, sand and dust. Figure 3 shows a picture of the largest sediment particles. These data indicate a large proportion of organic matter. The research showed it to contain about 90 % in this fraction. It consists mainly of organic residues. Elemental analysis shows the advantage of carbon. Figure 4 shows the results for sand fractions where the sand grains are quite visible and the elemental analysis points to the silica contents. Figure 5. shows the results for the smallest fraction which consists primarily of mineral matter and dust.

Conclusion

Analyzed sediment came from urban storm water drainage. Granulometric analysis showed the presence of ten specific factions, including the tree main: gravel, sand and dust. Research of mineral and organic sediment as a whole showed a relatively large proportion of mineral matter which is also confirmed by carrying out elemental analysis.

The research shows the great diversity of the sediment and analyzed the correlation between grain size and content of mineral and organic substances. Prevalence of dust fraction is also connected with the nature of the catchment and its location.

References

[1] *„Operat wodno-prawny na odprowadzanie wód opadowych lub roztopowych po oczyszczalni zlokalizowanej przy ulicy Aleje IX Wieków Kielc na kolektorze Si 9"*

[2] WOLSKA L.: *„Wynik analizy a jakość informacji środowiskowej. Opracowanie monograficzne z uwzględnieniem prac własnych"*, Zeszyty naukowe Politechniki Gdańskiej, Nr 608, Chemia LVII, Gdańsk 2008.

[3] MYŚLIŃSKA E.: *„Laboratoryjne badania gruntów"*, Wydawnictwo Naukowe PWN, Warszawa 1998.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

The Impact of Technical Graphite on the Structure and Compressive Strength of Calcium Silicate Products

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Abstract. Sand-lime products, also known as calcium silicate bricks are building materials characterized by high resistance to environmental factors. As one of the few products on the market, their average compressive strength reaches around 15 MPa. Moreover, they are resistant to biological and chemical corrosion, which consequently has a major impact on the living comfort in buildings built with calcium silicate bricks. Undoubtedly, they are eco friendly materials. Their heavy weight is an advantage with respect to sound absorbability features. Thus, the main objective of this research is determination of the phase structure and compressive strength of both original and modified sand-lime product. This paper will present original and modified products of increased compressive strength. In order to achieve the required analysis the original materials have been mixed with technical graphite followed by their comparison with 'pure' calcium silicate bricks. For the purpose of this study, the main judgment criteria of modified products have included compressive strength and SEM analysis.

Keywords: Sand-lime products, compressive strength, SEM, technical graphite.

1. Introduction

Sand-lime products, commonly known as calcium silicate bricks are construction materials of significant compressive strength. They are often used in bricklaying of external and internal construction walls, partition screens, fences, facing elevations etc. One of their main features includes a high resistance to environmental conditions. Calcium silicate materials are popular in many European countries and, to a lesser extent, other parts of the world. Sand-lime products are friendly to both residents and environment. Their resistance to biological and chemical corrosion improves the quality of living in such building. Furthermore, their significant weight has a major impact on their sound absorbability features [1,2].

Certainly, the manufacturing of sand-lime products is not a complex process. Calcium silicate bricks are obtained through mixing quartz sand, quicklime and a small amount of water. Such mixture is treated under pressure along with high temperature. This procedure is commonly known as the autoclave process. High pressure has an impact on the strong bond between silica and lime which results in calcium silicate $CaSiO_3$, followed by the chemical reaction that leads to the calcium carbonate $CaCO_3$. In the course of chemical reaction, lime binds to dioxide silica, which is the main sand ingredient used in the production process of calcium silicate bricks. The above reaction occurs in the 200^oC temperature [3].

The main objective of this research is to determine the phase structure of original sand lime products and those modified with technical graphite.

Technical graphite is a cluster of small monocrystals which demonstrates strong mechanical resistance to compression and, slightly lower, to stretching and shear. Accordingly, one may say that it is a hard and fissionable material [5,6].

The aim of conducted experiments is to determine the level of impact of introduced technical graphite on compression for the production of sand-lime bricks. It would be valuable to consider further introduction of the 'new' products to the building industry.

2. Methodology of Experimental Examinations

The experiments included two types of examinations. The former considered the scanning electron microscope (SEM) examination. The results of the analysis were demonstrated in the photographs and graphs showing composition and the amount of elements in a particular point of the material. The latter was carried out to establish compressive strength of ‘new’ sand-lime products.

Therefore, the experiment included two types of samples i.e. an original sand-lime product and a ‘new’ (modified) one with added technical graphite. The samples have been produced in the Calcium Silicate Production Plant according to the technological specifications.

The examinations were carried out in order to diagnose the phase structure of the abovementioned products and determine compressive strength of the ‘new’ materials.

3. Scientific Research

Lime in sand-lime products acts as a mineral bond, while sand and water are fundamental ingredients. The obtained mixture is treated under high pressure with increased temperature. High pressure has an impact on the strong bond between silica and lime which results in the calcium silicate $CaSiO_3$, and consequently as a result of chemical reaction the calcium carbonate $CaCO_3$ appears. In the course of chemical reaction, lime binds to dioxide silica SiO_2, which is the main sand ingredient used in the manufacturing of calcium silicate bricks. The described reaction, presented in the formula below, occurs in the 200^{o}C temperature:

$$2\ Ca(OH)_2 + SiO_2 \Rightarrow 2\ CaO \cdot SiO_2 \cdot 2\ H_2O \qquad (1)$$

The analysis of original calcium silicate products demonstrated the existence of the C-S-H phase. Formula **C**aO - **S**iO$_2$ - **H**$_2$O emerges as a result of the chemical reaction between calcium silica and water. Particular elements of such formula fall into the category of the most common elements found in the environment. Used water, according to current division, appears in 3 forms: chemical water in the form of C-S-H; water in gel and capillary pores; free water [4].

The impact of water is extremely important during the formation process of individual phases, not only in concrete but, as demonstrated by the SEM analysis, also in calcium silicate materials.

The purpose of the conducted experiments was to diagnose compressive strength of original and ‘new’ sand-lime products.

4. Results

The analysis of original calcium silicate products, which was performed by a scanning electron microscope, indicated the existence of the C-S-H phase (Calcium Silicate Hydrates) as well as tobermorite – the advanced form of C-S-H. The results are presented below in the photographs (fig 1, 2). The SEM photos show a sand grain SiO_2 surrounded by the amorphous C-S-H phase, which consequently transfers into tobermorite. The latter appears in the form of long and blunt plates (fig.2).

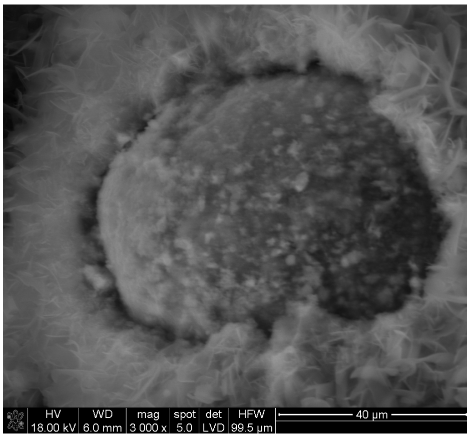


Fig. 1. Sand grains coated by the C-S-H phase.

Fig. 2. The phase structure of a sand-lime product.

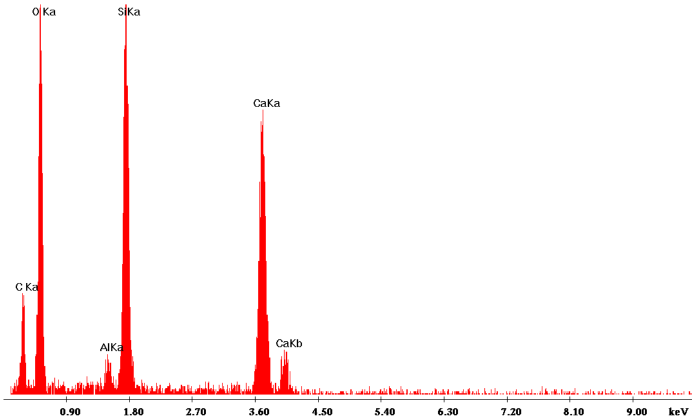


Fig. 3. Elemental structure of sand-lime products (point 1).

The presence of other elements is a result of specific ingredients acquisition to the calcium silicate mixture.

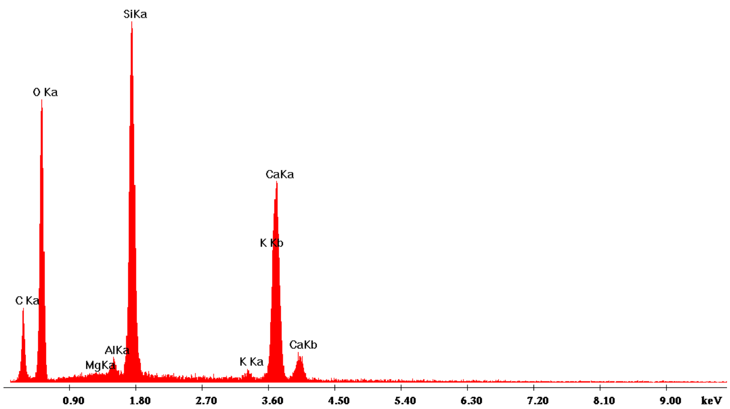
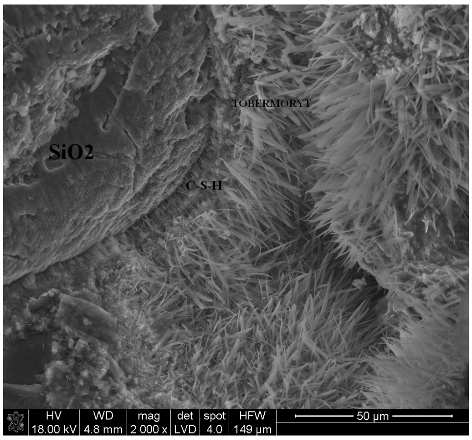


Fig. 4. Elemental structure of sand-lime products (tobermorite).

The added technical graphite to the production process of sand-lime products has caused the increase of compressive strength of the materials. It has also changed their structure. In ‘new’ products, it has been possible to notice the C-S-H phase and ‘impure’ tobermorite. The stripes of tobermorite have been covered with small particles. Moreover, tobermorite itself has had slightly changed shape as in the form of thinner stripes; however it has not been possible to classify it to the xonotlite phase yet. Increased strength is therefore, related to the shape of the phase next to the amorphous C-S-H (fig. 5 and 6).



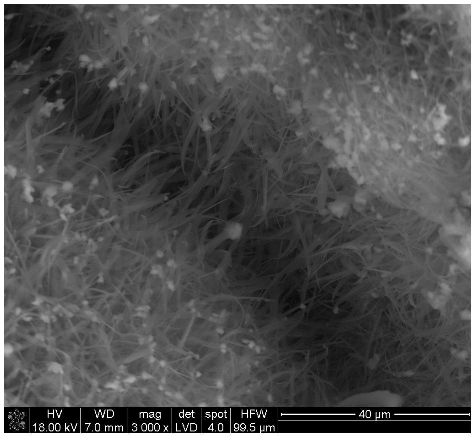


Fig. 5. SiO_2, C-S-H. and tobermorite in the 'new' calcium silicate product.

Fig. 6. Tobermorite in the modified sand-lime product.

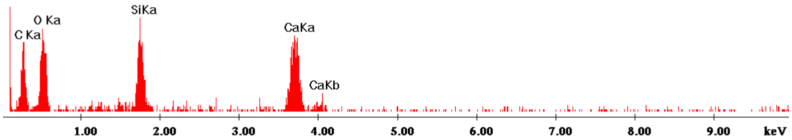


Fig. 7. Elemental structure of the modified sand-lime product.

Changes in compressive strength values are presented in the table below:

Ref. no.	Technical graphite [%]	Sand-lime mixture [%]	Compressive strenght [MPa]	Colour
1	WP - 0	100%	20,51	Light milk
2	0,22	99,78	24,04	Light grey
3	0,5	99,5	47,52	Dark milk
4	1,0	99,0	50,72	Grey milk
5	1,5	98,5	39,05	Grey milk
6	2,0	98,0	30,63	Dark grey

Tab.1. The values of compressive strength of original and 'new' sand-lime products.

5. Conclusions

Insertion of technical graphite to the sand-lime mixture increased the compressive strength of the 'new' product.

Technical graphite in the phase structure of calcium silicate materials led to shape and form modifications of tobermorite compared to its equivalent in the original product.

References

[1] DACHOWSKI, R, LINCZOWSKI, C. *Porównanie najpopularniejszych ściennych materiałów budowlanych stosowanych w budownictwie jednorodzinnym.* Przemiana. Kielce, 1998.

[2] LINCZOWSKI, C., DACHOWSKI, R. *Silikaty.* Zakład Produkcji Silikatów LUDYNIA Sp. Z o.o, 2001.

[3] Dachowski R., Stępień A. Masa do produkcji wyrobów silikatowych o podwyższonej wytrzymałości na ściskanie. Zgłoszenie patentu P.393518, 2010.

[4] Nocuń-Wczelik W. „Struktura i najważniejsze właściwości wybranych uwodnionych krzemianów wapniowych". Akademia Górniczo-Hutnicza, Kraków 1999.

[5] Patent SU 1411316 A1 – Mieszanka surowcowa do wykonania cegły silikatowej.

[6] Patent RU 2306290 – Sposób optymalizacji procesu technologicznego produkcji cegły silikatowej.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Analysis of the Behavior of Steel-Concrete Composite Truss

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Abstract. The design specifications of composite truss are included in the American standard (ASCE), however, in the European standards (Eurocodes), these construction systems are not covered. A finite element model (FEM) is developed using the software CAST3M to investigate the behavior of the composite trusses. The influence of various parameters, such as the diameter of the shear connectors, the degree of connection and the top chord section on the behavior of the composite truss and the shear connectors are analyzed. The model shows that the shear connection in the steel-concrete composite truss reduces its deflection by approximately 50 % in comparison with the steel truss. The significant influence of the top chord section on the shear forces in the shear connectors was observed.

Keywords: FEM Analysis, Top Chord Section, Shear Connectors, Shear Force.

1. Introduction

Composite steel-concrete construction is one of the most economical systems for building and bridge floors. To mobilize the efficiency of concrete in compression and steel in tension it is necessary to prevent the relative slip between the concrete and the steel element using the shear connectors. Different types of shear connectors are used nowadays. This concern the welded headed studs, the Hilti brackets and the welded perforated shear connectors [1] [2]. With the use of precast concrete slab or to develop composite action in non-composite structures, the use of the bolted shear connection is possible [3] [4].

Composite systems give the possibility to get spans till 20 m and the composite trusses are appropriate to meet the requirements for building height limitation, the need to run complex electrical, heating, ventilating, and communication systems and the even greater spans.

Since the mid 1960's to present, many investigations have been made in testing composite trusses mainly in USA and Canada, summarized in references [1] and [5]. The experimental results led to design recommendations and specification of the American Society of Civil Engineers (ASCE).

In Eurocode, there is no particular recommendation for the design of composite truss, except the formulas for the local effect of a concentrated longitudinal force and the distribution of the longitudinal shear force into local shear flow between steel section and concrete slab (EN 1994-2: 6.6.2.3). In fact, the forces are introduced into the concrete slab only in the positions of the increase of the axial force in the chord, i. e. where the filling bars are connected to the compressed chord (panel points).

In this paper the influence of the degree of connection, represented by the connector diameter, and the top chord section is analyzed regarding the stiffness and the resistance of the beams and the shear forces in the connectors. The analysis is based on the finite element modeling.

2. FEM Model

For the analysis of the composite truss the software CAST3M is used. The geometry of the truss (Fig. 1) is chosen from the references [1] [2]. The top chord of the steel joist is designed as ½ IPE 220, the bottom chord is a rectangular hollow section RHS 60×60×4 and the web members is a RHS 50×50×3. The top chord of the truss is connected to the concrete slab (1500×80 mm) with headed studs connectors (ϕ = 19 mm). All the components of the composite beam are modeled using beam elements with appropriate cross section. The web member elements are considered pin-ended but the chords are continuous.

The analysis was performed using the characteristic values of material properties. Simplified stress-strain diagrams of steel (S355) and concrete (C25/30) are shown in Fig. 2. The non-linear behaviour of the shear connection was modelled using beam elements uniformly distributed with a regular spacing equal to 100 mm along the span and located between the neutral axis of the top chord and the concrete slab [6]. In the model, virtual elastic-plastic material is used for the beam element in bending to represent the behaviour of the connection in shear. The uplift effects of the concrete slab are neglected in the model.

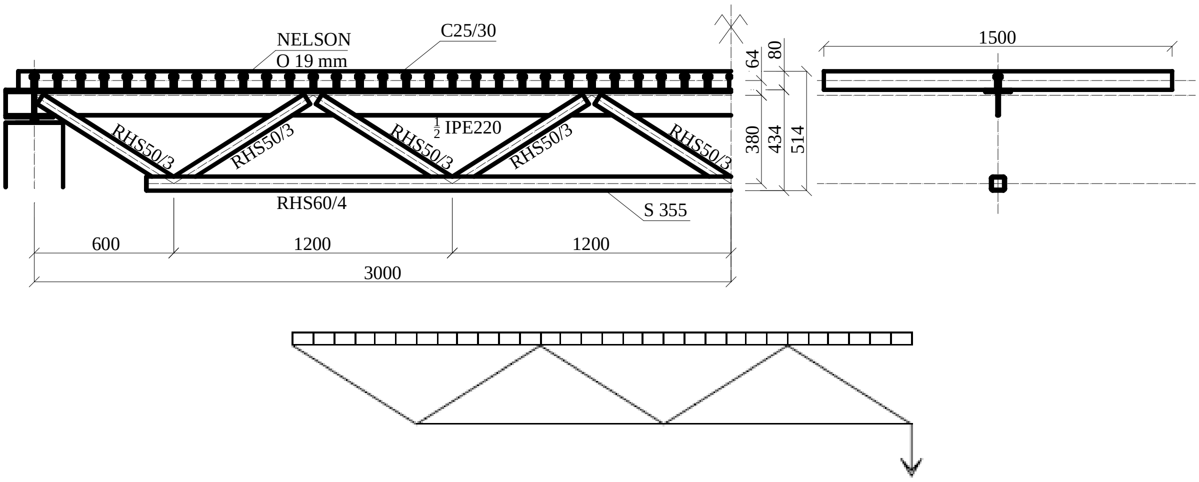

Fig. 1. Geometric characteristics of the composite truss and the FEM model.

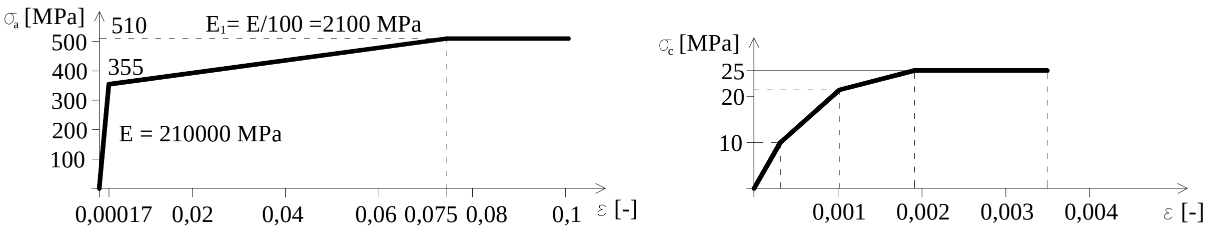

Fig. 2. Stress-strain curves of the steel and the concrete used in the FEM model.

As the results of the push-out test of shear connectors were not available, the formulae of Ollgaard [7], based on the results of the push-out tests, were used (1).

$$P_{max} = 0{,}336 A_d \sqrt{f_{ck} E_{cm}} \qquad (1)$$

The analytical expression of the evolution of the load – slip ($P_i - s_i$) curve is given by (2).

$$P_i = P^{0{,}4}_{\max\left(1-e^{-0{,}709 s_i}\right)} \qquad (2)$$

Loading of the truss was imposed at the central node of the bottom chord (Fig. 1). For elastic analysis, a load of 10000 N is applied and for plastic analysis the load is applied with a displacement control with a maximum value equal to (100 mm).

3. Results

In elastic analysis, the influence of the connectors is analyzed considering theoretical values of diameter from 0,1 mm to 100 mm. The influence on the stiffness of the composite truss is shown in Fig. 3. It can be observed that the usual diameter of 19 mm is enough to obtain a full connection in the truss. The Fig. 3 shows that the composite effect obtained by the shear connector increases the stiffness of the truss by a ratio of approximately 2 between the truss with no connection and full connection.

To analyze the effect of the top chord section on the composite truss, including the area *A* and the moment of inertia *I*, it was changed from ½ IPE80 to ½ IPE600 (18 different sections). The evolution of the distance between the centroid of the top chord and that of the whole truss, when the top chord parameters *A* and *I* increase, increases with the elastic analysis and decreases with the plastic analysis (Fig. 4). However, the global contribution of the top chord, on the whole section, remains weak and can be neglected.

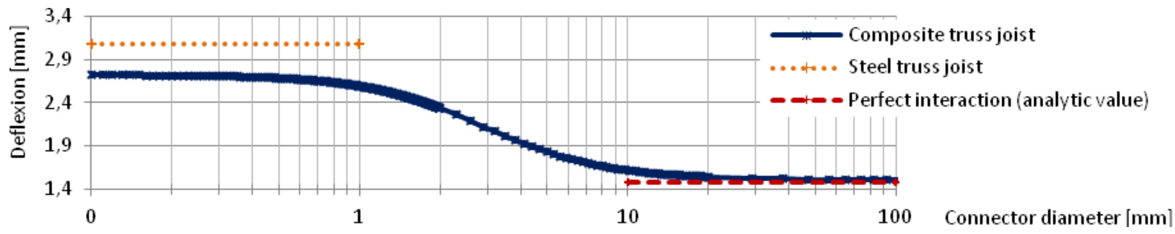

Fig.3. Influence of the connector diameter on the deflection of the composite truss.

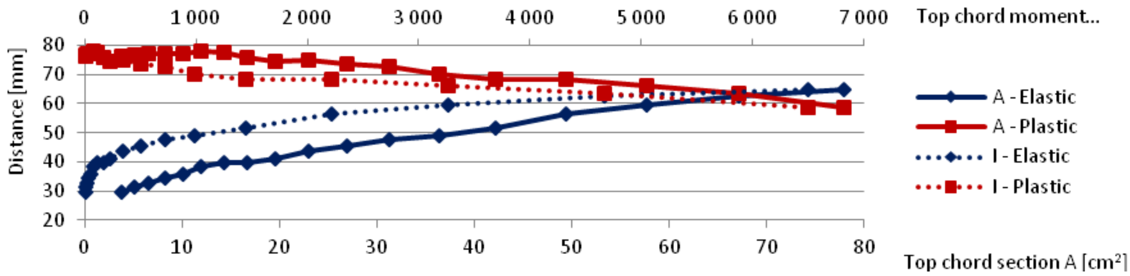

Fig.4. Distance between the top chord and the composite truss centroids (ϕ 19 mm connectors).

In Fig. 5 it can be seen that the influence of the top chord section on the stiffness of the truss decreases with the increase of the degree of connection for all the degrees of connection. This influence is more significant for low degrees of connections (no connection or partial connection) with small chord sections (lower than 15 cm^2 $1.5.10^{-3} m^2$). However for the real (and full) connection, the top chord section does not influence the stiffness of the composite truss significantly. The curves in Fig. 5 are drawn on the basis of the results of Fig. 3 where the full connection is represented by connector diameter of 50 mm, the real connection represented by the diameter of 19 mm and the partial connection by the diameter of 3 mm.

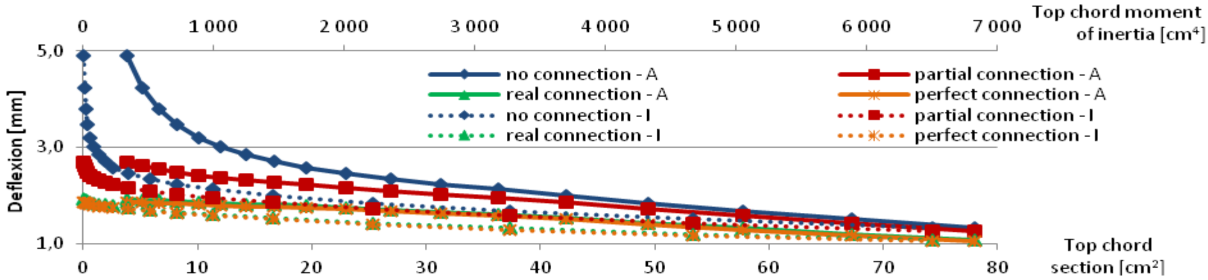

Fig.5. Influence of the degree of connection on the stiffness of the truss.

The distribution of the shear forces in the connectors along the beams are shown for different top chord sections with a displacement equal to 100 mm with plastic analysis (Fig. 6). It can be observed that the plastic deformation of the connectors gives uniform distribution of shear forces along the beam. This phenomenon is influenced by the ratio of resistance between the connector and the top chord section. Thus, it is necessary to optimize this ratio. Otherwise, the connectors in the panel points will transfer the main part of shear forces.

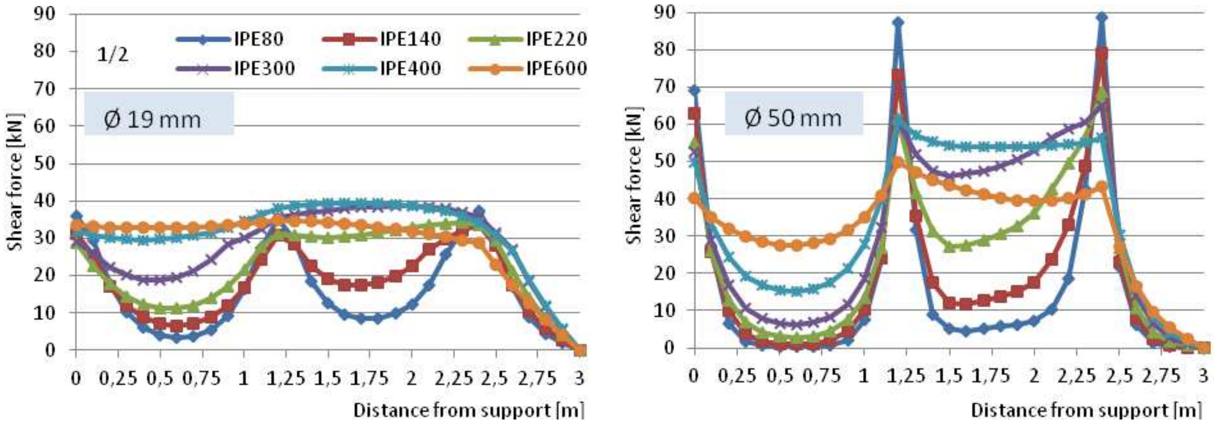

Fig.6. Influence of the top chord section on the shear forces in the connectors (Ø 19 mm and 50 mm).

4. Conclusion

The influence of different parameters of the steel-concrete composite truss on its behavior was investigated. The parametric studies showed that the top chord section has a non significant effect on the flexural stiffness and capacity of the composite truss, because it is usually located very near to the neutral axis of the composite member, in elastic and plastic analysis. However, the top chord section has a significant influence on the shear forces in the connectors. In fact, the ratio between the characteristics of the shear connector and the top chord section governs the distribution of shear force along the beam.

The study showed that the main function of the small top chord sections is rather to provide an attachment surface for the shear connectors. Therefore the investigation will be focused on the composite truss beams without top chord, where the web members are connected directly to the concrete slab. Another aspect concerns the proposal of analytical formula to predict the distribution of shear forces in the connectors for various ratios of shear connectors and top chord sections.

References

[1] MACHACEK, J., CUDEJKO M. *Shear connection in steel and concrete composite trusses,* SDSS'Rio 2010 Stability and Ductility of Steel Structures, Rio de Janeiro, Brazil, September 8 – 10, 2010, 8 p.

[2] MACHACEK, J., CUDEJKO M. *Longitudinal shear in composite steel and concrete trusses,* Engineering Structures 31, 2009, p. 1313-1320.

[3] SCHAAP, B. *Methods to Develop Composite Action in Non-Composite Bridge Floor Systems: Part I,* Master Thesis, The University of Texas at Austin, 2004.

[4] HUNGERFORD, B. E. *Methods to Develop Composite Action in Non-Composite Bridge Floor Systems: Part II,* Master Thesis, The University of Texas at Austin, 2004.

[5] SAMUELSON, D. *Composite steel joist*, Engineering Journal, 2002, p. 111-120.

[6] BUJNAK, J. *Global analysis of steel-concrete composite beams - Analytical approach and nonlinear modelling,* PhD Thesis, Blaise Pascal University, Clermont-Ferrand II, France, 2007. [In French]

[7] OLLGAARD, J.G., SLUTTER R.G., FISHER J.W. *Shear strength of stud connectors in lightweight and normal-density concrete,* AISC Engineering Journal, 1971, vol. 8, n°2, p. 55-64.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Implementation Balanced Scorecard Methodology in Civil Engineering

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Abstract. The article deals with how is the modern trends in implementing balance scorecard methodology in the construction business also all civil engineering. Detail written how to understand the methodology and how to apply it correctly. It also describes how this methodology could help in the direction of modern trends not only in civil engineering.

Keywords: Balanced scorecard, balanced scorecard methodology, civil enginnering.

1. Introduction

The decisive factor for the success of any company in the information age is the ability, how can mobilize and utilize the available tangible and intangible assets. With companies now want to survive and succeed they should invest in intellectual assets and to use control and measuring systems which are derived from their strategies and capabilities. Of business has undergone a significant change in the last century.

While during the industrial era was accomplishment of company dependent skill demands, how can use technological new means to effectively produce most standard products entering the information age, many source claims paying competitive environment for the industrial era, overcome. Managers today need a business management toolkit or some means by which to evaluate various aspects arising from the business and its environment and watch the business forward in achieving the objectives set. This toolbox provides managers so. Balanced Scorecard, which they can navigate their future success.

2. History of the Methodology

Explanation of the methodology of Balanced Scorecard can be an initial transfer of the word. The word "balanced" can be translated as compensated and in fact represents a balance between financial and non-financial indicators, the objectives of short and long term, the late and preliminary indicators and between internal and external performance factors. The word "scorecard" has origins in American sport, where a list of exactly who has contributed to the outcome of the game and how, respectively did not.

Balanced Scorecard is a strategic performance measurement system, which was at the beginning of the 90th years due to U.S. experts Robert Kaplan and David Norton. The principal change brought about by the Balanced Scorecard, the expansion and integration of enterprise performance measurement for purely financial indicators to indicators from other perspectives of business. The targets and indicators based on the vision and business strategy.

The Balanced Scorecard is to achieve a comprehensive "balanced" and in several directions: between the short and long-term objectives, value and kind of indicators among the late indicators and drivers, including internal and external performance factors. [1]

It is a method in which the company evaluates the performance of the four main groups of indicators: financial performance, results and performance to market to customers, results and performance of internal business processes and results for the development of employees and intangible assets. It is a strategic business performance measurement system. It handles the vision and strategic targets in the comprehensive set of indicators. [3]

The concept of Balanced Scorecard has become a landmark in the criticism of some explanatory ability of value (financial) criteria for measuring business performance and to assess the success of its survival in the future. Criticism of the traditional indicators of company performance lies in the fact that most traditional indicators are based on accounting information, particularly on financial income. Their biggest drawback is that factor taken into account the risk of inflation, do not engage in time for the money. Does not compare to the profit opportunity costs. Financial indicators play a major role in the measurement of results-oriented business performance. Also, analysis of financial performance is the basis for the analysis of non-financial factors affecting the company's strategic success.

The company can be successful without a strategy and strategic planning process. Balanced Scorecard is not a strategy, it is a managerial role focusing on financial and non-financial goals, it is useful to communicate strategic goals and objectives of business units at all levels of the organization, continuing assessment process involves. Balanced Scorecard provides feedback to improve internal processes. The financial targets that tell managers what had happened, they are poor indicators. Managers need to know whether the future business success. Future success depends on non-financial objectives "leading indicators".

3. Balanced Scorecard as the Current Strategic Business Performance Measurement System

The company which has a future-oriented leadership with a common perspective, able to develop a vision and is linked to the collective spirit. The strategy in this sense is an idea, concept and culture, supported by the company and all staff is oriented to the future. [5]

Significant impact on achieving future success is appropriately selected and implemented the strategy of the company. Using the Balanced Scorecard reporting company becomes more complex. The reporting of financial and non-financial and market data providing investors with more relevant information, making the company more transparent and more attractive to investors. Balanced Scorecard can induce innovation and dramatically save cost and help us feel more rapid market opportunities. To determine the weighted indicators is necessary to consider not only financial card (such as overheads, profit margin, return on assets, the share price...), but mostly business vision, connection to customers - loyalty, the continuous improvement, innovation, quality, staff qualifications, etc. This structure allows you to track how companies develop their strategic assets necessary to achieve the objectives. The answers to the questions, the company is able to formulate an appropriate strategy.

Balanced Scorecard is thus a tool for effective strategic management company that is recognized as the best approach to transform corporate vision into concrete measurable actions, which take into account the factors that create long term value. This approach responds to the requirements of time and helps businesses integrate strategy into everyday decision-making process of their workers, thus promotes synergies and significantly increases the efficiency of the company and its competitive ability. [3]

Very useful for the method is that it is no respecter of setting strategic directions and objectives only in terms of financial but takes more views. Balanced Scorecard provides a framework for language communications company mission and vision and overall strategy. It is used to enable staff to be aware of the main contexts that affect current and future success.

When using the Balanced Scorecard is a prerequisite for measurement of individual indicators. Some, including financial indicators are easily measurable. All necessary information is obtainable from the accounts. Measurement indicators in the perspective of internal processes or in the perspective of learning and growth it may be difficult. It is not common practice for companies that collect such data to target and pursue. It is therefore essential need for monitoring and collecting data to create. [4] Balanced Scorecard is not a static model. After its introduction should monitor and analyze all relevant information to verify the model. In case of irregularities, the company responsible for keeping adequate instruments or change the selected activities.

4. Why Focus on the Balanced Scorecard

The Balanced Scorecard is a recognition that financial and non-financial indicators should be part of a comprehensive information system available to employees at all corporate levels. Employees at the top positions must understand the financial consequences of their own decisions and actions, senior management must understand the forces drove to ensure long-term financial success. Targets and indicators for the Balanced Scorecard are derived from the vertical process of the mission and strategy of the company. Balanced Scorecard should transfer mission and strategy into tangible business plans and indicators. Balanced Scorecard is more than tactical or operational system parameters. Innovative companies use this methodology to manage their long-term strategy and measuring its properties used to make critical management processes:

- to clarify a design vision and strategy into specific objectives,
- to communicate and link strategic plans and indicators,
- planning and setting targets and align strategic initiatives,
- to enhance strategic feedback and learning process.

Balanced Scorecard process starts with senior management team work. In setting the financial goals the team needs to consider whether to focus on market growth and turnover, profitability or cash flow creation. The customer perspective of the management team must be accurate in selecting customer segments, which decided to compete. After defining the financial targets set firm targets and indicators for their own internal processes. This is one of the major benefits of innovation and the Balanced Scorecard. Traditional performance measurement systems, even those used by many non-financial indicators, focus on improvement of cost, quality and duration of the cycles of existing processes. Balanced Scorecard focuses on processes to achieve reversal in performance for most customers and shareholders. Often this will reveal a completely new internal processes in which a company must achieve a perfect result, that his strategy was successful. The concept of Balanced Scorecard and logic is not new. What is new is a simple, clear design and more formalized process of performance management and linking strategy with performance measurement and outcomes.

5. Successful Implementation of the Balanced Scorecard

Before the strategic planning process includes identifying multiple strategic objectives for depositories, we started measuring them, setting targets, assessing performance and using feedback to improve in real time. Balanced Scorecard is a simple tool to achieve those aspects of performance management and strategic planning process.

Top and senior management support:

Senior management and middle management must take the Balanced Scorecard to be managed from top to bottom across the entire organization. It is important that senior and middle managers fully understand the concept and process of Balanced Scorecard. Managers should be trained through seminars and workshops. The role of the CEO is much more critical in the success of the Balanced Scorecard. He, respectively. she should take the lead in the introduction and implementation of the Balanced Scorecard. A number of organizations began with the Balanced Scorecard from senior management and CEO, and then follow the cascade down to other levels of the organization.

The identification of critical success factors.
Translating critical success factors into measurable targets (meters).
Linking pay to performance measurement.
Installing a simple tracking system.
Creating a Balanced Scorecard links at all levels of organization.
Communication.
Linking strategic planning, Balanced Scorecard and budgeting process.

6. Creating and Implementing Corporate Balanced Scorecard

Creating Corporate Balanced Scorecard is a systematic process. It allows the creation of consensus and clarify how to transfer the mission and strategy of each company's strategic goals. Plays an important role of top management teamwork. The process of Balanced Scorecard consists of the following steps:

- clarification of strategic objectives,
- link the strategic objectives of chains of cause and effect,
- selection and design parameters,
- setting targets,
- adoption of strategic objectives. [2]

The processing of these steps forms the core of the implementation of the Balanced Scorecard. By creating a Balanced Scorecard to create a new framework for evaluating and rewarding employees according to their contribution to the achievement of business objectives. Strategic Objectives and Balanced Scorecard indicators are communicated across the enterprise through business newspaper, video or electronically using the business portal. These messages inform employees about the critical objectives to be achieved, if the successful business strategy. If employees understand the highest goals and indicators can be set local targets that support the overall company strategy. Balanced Scorecard also provides a basis for communication and sharing of responsibility for executive managers and board for the implementation of corporate strategy. It encourages dialogue between business units, managers and directors, for not only targets, but also the formulation and implementation of a strategy enabling a further increase in performance.

7. Conclusion

A prerequisite for the drafting of the implementation of the Balanced Scorecard is to acquire knowledge about the Balanced Scorecard methodology of available resources, consisting primarily of professional foreign literature and information from websites. In this article I have tried to show how this methodology can help us.

References

[1] FIBÍROVÁ, J. – ŠOLJAKOVÁ, L.: *Value Management tools and measuring business performance*. Praha 2005. ISBN 80–7357–084–X.

[2] HORVÁTH & PARTNERS: *Balanced Scorecard in practice*. Praha 2004. ISBN 978–80–7259–033–9.

[3] KAPLAN, R. S. – NORTON, D. P: *Balanced Scorecard.* Praha 2000. ISBN 80–7261–032–5.

[4] BRABECK, R. – ŠTOFKOVÁ, K.: *Balanced Scorecard: ein kurzer Überblick über ein Innovatives Instrument im zielorientierten Management. In: Crisis management.* – Year. 5, č. 1 (2006), s. 13–16. ISSN 1336–0019.

[5] SLÁVIK, Š.*: Strategic management*. Bratislava 2005. ISBN 80-89085-49-0.

[6] SCHMALL, T.: *Balanced Scorecard as a managed tool*. Productivity. no. 6 (2004), s. 2–3 ISSN 1335–5961.

[7] NIVEN, P. R.: *Balanced scorecard step–by–step for government and nonprofit agencies*. N.J. : John Wiley & Sons, 2003. ISBN 0–471–42328–9.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Railway Superstructure Stabilization with Geosynthetic Layers

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Abstract. The paper deals with the research and development related to investigation of geogrid railway ballast reinforcement at the Department of Transport Infrastructure and Municipal Engineering of Szechenyi Istvan University. It summarizes the theory of the geometrical deterioration of railway tracks, as well as the advantages of the use of geogrid reinforced ballast in railway superstructure. This article summarizes the results of the field tests with five different geogrid types on a Hungarian main railway line and laboratory multi-level shear box tests. It points out to future research possibilities, for example the modelling of laboratory multi-level shear box tests with discrete element method that may certify their results.

Keywords: Railway, superstructure, stabilisation, geogrid-reinforcement ballast.

1. Introduction

1.1. The Geometrical Deterioration Process of Ballasted Railway Tracks

Geometrically and structurally perfect railway tracks can't be constructed because of the tolerances and quality differences of constructional elements as well as inaccuracy of geodetic alignment and technology during construction and maintenance works. However the railway tracks' geometry and quality differ from the accurate conditions, but if the tolerances and quality differences are below their permissible limits, the tracks can be opened to traffic for the speed limit in accordance with the appropriate acceptance requirements.

The train traffic and its damaging effects generate harmful changes in the railway track; this is actually the geometrical deterioration process. This geometrical deterioration takes place by strict physical laws and it is an irreversible process. Its speed can be influenced as well as decreased by maintenance works, but it can be never stopped.

The function of geometrical deterioration process in explicit form is the following:

$$C = C_0 \cdot e^{\alpha \cdot m \cdot v^2} . \tag{1}$$

In (1) "C" means the geometrical quality of the railway track, "C_0" is a parameter related to the initial track condition and it is able to show the maintenance's quality, "m" is the through-rolled mass, "v" is the equivalent speed, and "α" is a superstructure dependent parameter.

In the exponent of "e" in (1) there is an expression related to the energy of motion [1].

Improvement (i.e. reduction of "C") can only be reached with maintenance works (e.g. tamping work). "C" value can be decreased more in a track with worse geometrical quality but the result will be able to be characterized with more and more "C" value.

1.2. Function and Deterioration of Railway Ballast

The railway ballast has to support the track stably and flexible and it has to distribute the load from the sleepers' lower faces to the substructure (embankment and/or supplemental layer). Ballast material should have adequate resistance in longitudinal and transversal directions which are necessary for the track's bedding and structural stability. The direction, the settlement and plane distortion values have to be ensured in respect to relative geometry.

The good quality ballast material is an aggregation of non-cohesive, graduated and cubic shaped, angular particles. In this particle aggregation the vertical load of vehicles is distributed through the 'stone-skeleton' to the lower layers, while the horizontal loads are balanced as passive earth pressure. In both load-distributions the interlocking effect between particles is very important.

External effects (mainly the repetitive through-rolled axles, as well as the weather) change the behaviour of particle aggregation; therefore the actual geometry of the track will be worse and worse. The ballast particles can be pushed into lower layers especially in the case of weak embankment and/or supplemental layer, therefore large vertical plastic deformations (settlements) can arise.

Because of the reasons above, a structural modification seems to be practical for prevention to ensure that particles of the aggregate act together. Using geogrid reinforcement under the ballast bed can be a good solution for this problem.

1.3. Effect of Geogrid Reinforcement under the Railway Ballast

Fig. 1 illustrates the interlocking effect which is typical of geogrids. Acting together between the granular and angular ballast material particles and the geogrid is the basis of the increase of the internal shear resistance of the layer-structure.

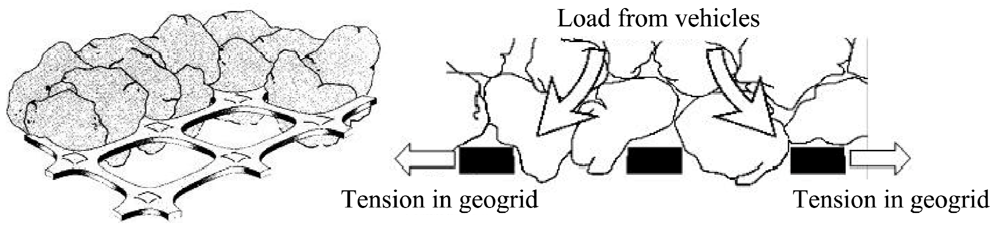

Fig. 1. Interlocking effect and load-bearing of a geogrid-reinforced ballast [2].

The particles can penetrate into the geogrid layer's apertures, and grapple on to ribs. The other particles bear up onto this composite (particle and geogrid) layer. Its face structure is advantageous for the higher internal shear resistance. The re-arrangement of particles is hindered by the composite layer in vertical and horizontal planes too. Stresses arise in the ribs and junctions of the geogrid due to vehicle load, the geogrid can offer resistance against these stresses with tensile strength and low strain. Tensile strength should be adequate high, but failure strain should be acceptable low, because of the load bearing with small strain.

2. Goal of the Research Work

There are three possible methods for the investigation of the stabilisation effect of geogrid layers under railway ballast:

- constructing trial field track sections where the track geometry position and its changes are regularly measured,
- setting up laboratory tests which are adequate for determining parameters related to ballast material-geogrid interaction,
- computer-aided modelling for improving and generalizing the correlations.

At the trial field track sections, geometric levelling are done on the rail-heads, therefore the track's geometry changes can be measured and determined. If the measuring points on the rail-heads are close enough to each other and the measurements are often repeated, there will be a large database which can be processed. Values below and their changes can be determined as a function of through-rolled axle tons as well as of the elapsed time from the first tamping work after the geogrid's built-in:

- cross settlement difference between the two rail-heads,
- settlement (calculated with different chord lengths on one rail),

- plane distortion (twist) (calculated with different base lengths),
- settlement on individual sleepers.

The laboratory tests can help to better understand the behaviour of geogrid-reinforced railway ballast. A great deal of parameters influences this behaviour (e.g. geogrid type, properties and density of ballast material, depth of ballast layer, elasticity of support layer, etc.). The changing of interlocking effect in the ballast material as a function of vertical distance from geogrid layer is a key-role for the conformability of evaluation of geogrid reinforcement's use. The research team would like to get much information from the test of interlocking effect in order to evaluate the expedience geogrid-reinforced ballast.

Computer-aided modelling is needed for numerous reasons. Consideration of all variable parameters will unfeasibly extend the number of laboratory tests. If an acceptable number of tests have been implemented, models can be constructed whose behaviour can be confirmed and certified by the measured parameters. Using this model in computer-aided modelling is conformable for more detailed analysis of various parameters' effect, as well as for universalizing of statements which are needed for determining correct general laws about the behaviour of geogrid-reinforced railway ballast. This paper doesn't deal with computer-aided modelling of geogrid-reinforced railway ballast material in detail.

3. Tests and Results

The research team had an opportunity to create a 700 m long trial field track section with uniform soil properties on a Hungarian main railway line in May, 2010. This railway section contains substructure faults, e.g. local water pockets. Three different subsections have been constructed and five (PP and PE) geogrid types have been built-in during the ballast cleaning:

- sections without ballast cleaning just tamped,
- sections with ballast cleaning without geogrid reinforcement and tamped,
- sections with ballast cleaning with geogrid reinforcement and tamped.

Geogrid types 1, 3 and 4 contain geotextile layers too, types 2 and 5 are single geogrids. After the final tamping work of the construction (17th of June, 2010) several geometric levelling have been done and their data have been processed. Fig. 2 shows the mean and standard deviation of plane distortion (twist) as a function of elapsed days from 17th of June, 2010. On the 124th day there was a tamping work again, before and after it geometric levelling has been accomplished.

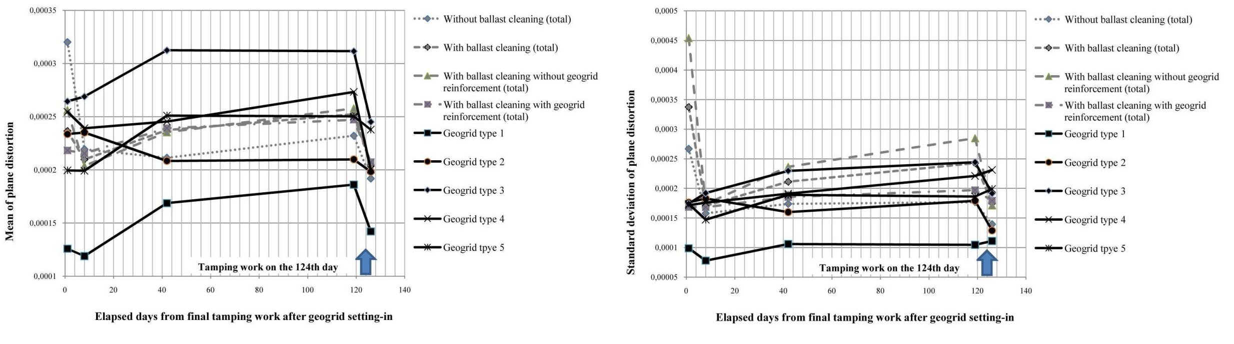

Fig. 2. Mean (left) and standard deviation (right) of plane distortion (twist) (3.6 m base length) as a function of elapsed days from final tamping work after geogrid setting-in.

In Fig. 2 it can be seen that sections without geogrid reinforcement are characterized with higher mean and standard deviation values than several geogrid types, e.g. geogrid type 1 and 2, but other types do not improve the track's geometric stability (in this controlled short time period). Sections without ballast cleaning just tamped do not show low values in the mean diagram because of the relatively dense ballast superstructure, but in the standard deviation diagram this section seems to be the worst.

The reference time before the 2nd tamping was not long enough, the result will be more precise with extended inspection time, so the measurements have to be continued in the future.

For the laboratory tests the research team planned and constructed a multi-level shear box with which the change of the interlocking effect in the railway ballast material was investigated as a function of vertical distance from the geogrid layer. This function in the above form hasn't been determined by anyone with lab test, international researchers got such kind of results with help of DEM modelling of geogrid pullout test [2] but not with a multi-level shear box.

The mentioned 1.0×1.0×1.0 m multi-level shear box contains max. ten 10 cm high horizontal frames filled with max. 50 cm high Thermopan layer, thereon 10 cm height sand, thereon geogrid layer with or without geotextile as well as thereon four times 10 cm height railway ballast material. Horizontal shear resistance can be measured at each frame connection. The tests up to now are related to only one geogrid type (geogrid type 2 in the field test) and to only new loose and dense ballast material. Tests were conducted with and without geogrid reinforcement too. Fig. 3 shows horizontal shear resistance of railway ballast aggregate.

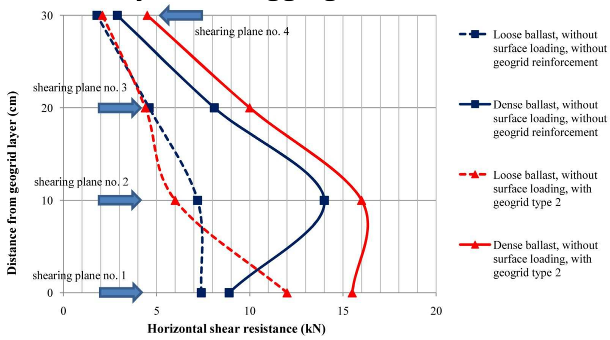

Fig. 3. Horizontal shear resistance as a function of distance from geogrid layer.

Dense specimens had higher shear resistance than loose ones, geogrid reinforcement increased these values especially in the near region of geogrid layer same like in [2].

4. Conclusion

It can be unequivocally stated that with adequate geogrid type the ballasted railway superstructure can be strengthened. More measurements have to be done in trial and in laboratory tests too, then the computer-aided modelling will be needed to check the results in laboratory multi-level shearing tests.

Acknowledgement

This paper is supported by TAMOP-4.2.1/B-09/1/KONV-2010-0003: Mobility and Environment. The Project is supported by the EU and co-financed by the European Social Fund.

References

[1] HUNGARIAN RAILWAYS *Modern railway–modern railway technique. Railway construction and maintenance, Vol. 2.* Budapest: MAV Rt., 1999, pp. 157-160

[2] KONIETZKY, H. et. al. *Use of DEM to model the interlocking effect of geogrids under static and cyclic loading, Numerical Modelling in Micromechanics via Particle methods* (edited SHIMIZU, Y. et. al.), Rotterdam: A. A. Balkema, 2004, pp. 3-11

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Indoor Air Plaquality Problems in the Context of the Analysis of Energy Savings Opportunities

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Abstract. The paper presents current problems of indoor air quality (IAQ) resulting from the implementation of new trends in constructions, the use modern finishing materials, equipments and changing patterns of use of facilities and functions. Issues associated with the potential to reduce energy consuming of treatment processes for air ventilation were also raised. The problem was recognized in the context of current legislation and trends in environmental policy and energy policy.
With regard to the raised subject, the paper presents the innovative technological solutions that can reconcile the problem of rising energy demand associated with the need for air treatment, along with the trend to save energy. The concept is based on an active indoor air purification system of radiant catalytic ionization technology.

Keywords: Indoor air quality, energy, thermal, analysis.

1. Introduction

The dynamic development of civilization and the new trends in building engineering changed the functioning of society. It's been said that in developed countries people today spend more than 90% of their time indoors. In this close space air creates a unique microclimate which has a different composition from the atmospheric air. For this reason, it is important to ensure adequate indoor air quality, which has a greater impact on human health than pollution in the outdoor air.

Simultaneously staying of the large concentration of people in closed space causing a risk of accumulation and spread of bacteria and virus microorganisms. In each of these places also around us an increasing number of electrical, mechanical and synthetic materials. Most of them have a significant impact on indoor air causing an air quality deterioration. Improving indoor air quality, depends on a number of processes that require high energy demand. The international strategy to combat energy consumption of all responsible units is implemented through the issuance of European Directive 2002/91/EU. This Directive enforces the need for priority answer to the problem of energy consumption. The Directive requires limiting the maximum energy demand for buildings in the European Union. For the Polish, adjustment the current state to the new EU regulatory guidelines, involves the significant reduction of energy consumption in buildings.

2. The Frequent Problem Areas with Indoor Air Quality

Study of chemicals air contamination in residential and public buildings are the subject of scientific research in different research centers and sanitations services around the world. In Poland, the units of the sanitary supervision perform analysis of the concentrations of chemical compounds in the close space. Unfortunately it mainly affects only the job close space.

The main sources of air pollutants are pathogen and chemical compounds, so-called volatile organic compounds. They include a variety of chemicals, inter alia: aliphatic hydrocarbon, aromatic hydrocarbon, esters, carbonyl compounds, phenol and its derivatives, moreover ozone, nitrogen oxides, and carbon dioxide.

Volatile Organic Compounds are emitted by a wide array of products such as:

- used building and finishing materials such as: paints, wallpaper, floor coverings
- office equipment such as copiers and printers,
- air pollution from supply air
- smoking

Based on previous research results is not possible to assess the quality assessment of chemical air contaminations in buildings across the country. Obtained results have not been the effect of the continuous monitoring. Over 65% analyses was performed mainly as a results of residents or employees complaints. The obtained image of indoor air quality in residential and public utility buildings, can be only approximate value to the real situation.

According to published national data of chemical air contaminations, show that too little quantity of fresh air and too high concentrations of chemicals may cause, among users, symptoms of sick building syndrome (SBS). SBS is a combination of ailments (a syndrome) associated with an individual's place of work (office building) or residence. It is characterized by symptoms such as: headache, dizziness, fainting, nausea, symptoms of fatigue, irritation of mucous membranes, breathing difficulties.

2.1. Analysis of the Possibility of Reducing Energy Consumption Necessary to the Air Treatment

It is estimated that about. 20-30% total building energy demand is used for ventilation and air conditioning systems. These installations are designed to supply and air treatment by diluting to the safe for humans health concentration levels of pollutants.

A significant percentage of energy consumption in these processes in the total energy demand of the building making the need to search solutions for reducing it. However you must remember that any action taken to meet new requirements may not cause a deterioration of indoor air quality.

The most energy consuming processes of outdoor air treatment is the thermo-moisture treatment in the air conditioning system. Well known possibility of reducing energy consumptions in this aspect does not take into account the volume of air flow and air quality. Performing this analysis, we should consider it in two contexts.

The first is treatment of the supplied fresh air. It is logical that you must consider the extend of possibility changing ventilations air flow, which has a significant impact on energy consumption for air conditioning and ventilations systems.

Second, it is necessary to reductions emissions inside room. While, use of low-emission construction materials is possible, but drastically reduce pollutant emissions by the person, not anymore. Reducing the concentration of harmful substances in the air, can be achieve through the use of modern technology of air purification. It could reduce necessity of dilution recirculated air by means of increasing volume of supply fresh air, which treatment is energy-consuming.

2.2. Current Legistlations

More current legislations requirements the indoor air quality is divided into two groups.

First, with the primary objective is the protection of health, assuming that some chemicals compounds and pathogens causing adwerse health effects. Regulations determine the maximum permissible maximum concentration of theese substance, which do not cause negative changes in state of healt.

Second, is designed to provide thermal comfort. On the basis of this criterion it is determined the intensity of the ventilation process. Here you can distinguish more particular procedures: procedure of volume requirements ventilation air flow, procedure of allowable concentration of pollutants, the procedure associated with counting the percentage of people dissatisfied [4] (ie the number of person which feel discomfort by staying in closed space during the analysis.) Both Poland and other countries are used only certain method of determinig an air quality, mainly depanding on the functions of a building.

Some of Polish regulations are outdated for the sake of changing utility function of building and used of modern materials and installations.

The list of current legislations:

- PN-78/B-03421: Ventilation and air conditioning system. Calculated parameters of indoor environmental for rooms intended for permanent human habitation
- Regulation of the Minister of Labour and Social Policy of 11 December 1989 on the maximum permissible concentrations of harmful factors in the workplace (Journal of Laws No. 114, item 495)
- Decree of the Minister of Health and Social Care of 12.03.96 on the permissible concentration and intensity of harmful factors, secreted by building materials, equipment and fittings in rooms designed to accommodate people-Polish Monitor of 22.04.96 No.19, 3
- PN-EN ISO 7730:2006(U) Ergonomics of the thermal environment -Analytical determination and interpretation of thermal comfort using calculation of the PMV and PPD indices and local thermal comfort criteria (ISO 7730:2005);
- PN-EN 15251:2007 Indoor environmental input parameters for design and assessment of energy performance of buildings addressing indoor air quality, thermal environment, lighting and acoustics

In relation to energy policy which promoted energy efficiency can be seen mainly in the evolution of standards to be met by the new buildings put to use. Thermal standards are currently several times more stringent than ever.

2.3. Modern Technologies for Indoor Air Purification

In order to obtain the required quality parameters, the ventilation air must be treated. The most available air cleaning technologies typically are traditional passive technologies, based on the mechanical, electronic air filters installed in incoming outdoor air and recirculated air streams. In contrast to products that treat odours with passive technologies, there are also active air purification technologies which work together to clean indoor air like nature cleans air outdoors.

One of the innovative, and active methods is an air purification using Radiant Catalytic Jonization (RCI) technology. This is a highly effective system designed to utilize germicidal, reduction quantity of airbone contaminations and odours. This technology makes use of the same oxidation and ionizing properties of light as naturally occurring sunlight, and consist in uses a specific high intensity UV light that reflects off of 5 rare and noble metals and this reaction with moisture in the air creates super oxide ions and hydro-peroxides which have powerful anti-microbial properties. The hydro-peroxides are basically hydrogen peroxide in vapor form.

The resulting blend of "friendly" ion-oxidants, produced by the RCI technology ensures effective reduction of contained in the air, the chemical, mechanical and biological pollutants. Moreover, it can treatment the air without excessive production of ozone.

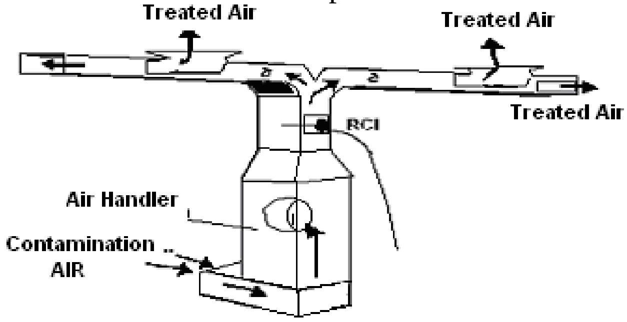

Fig. 1. Functional diagram of typical RCI units installation.

The units using RCI technology should be installed in any HVAC system directly into the plenum or directly above the air handler in remote or 'downstream' location. They work by converting water vapour (H2O) and Oxygen (O2) from untreated air into hydro-peroxides and hydroxyls, eliminating various microbes (odours, mould, bacteria, viruses, VOC's etc) creating a healthier indoor environment. Untreated air enters the air handlers, passes by the RCI unit and releases treated air through the AC vents.

3. Conclusion

In pursuit of energy-saving expressed by thermo modernization and using new technologies in construction are in accordance with the Energy Policy of the European Union. However most of this modernizations was badly conducted. In many cases, it cause disturbance of natural air change and significantly deterioration indoor air quality which cause adversely affect the human health.

This problem concern a large number of houses in our country, because during the thermo-modernization, everything has been designed, forgetting about the "air". We used airtight building to ensure low operating costs Moreover, we apply modern, durable and practical paint, wallpaper and carpeting to improve aesthetics and comfort of use of buildings. This approach, very often leads to that more biological and chemical contamination are trapped inside the closed space causing adverse health effects. In many new buildings there are mechanical ventilation which ensures adequate volume of fresh air supply. However, due to the increasing amount of pollution, it must be remebered that today volume of air change is minimally. Moreover the hygienic condition of ventilation system often cause development of toxic household fungi, bacteria, mites and other microorganisms.

These situations make it necessary to particullary pay attention to indoor air quality and searching innovative solutions and air purification technology.

Implementation of practical RCI air cleaning technologies for both may improve indoor air quality or enable IAQ levels to be maintained with reduced outdoor air supply and concomitant energy savings. The cost of energy use of efficient particle filtration is much smaller than the cost of energy used to treatment growing volume of ventilation fresh air.

Dimensioning this problem in terms of energy efficiency, will be the subject of further research.

References

[1] POŚNIAK M. NICOLE, R. *Zanieczyszczenia chemiczne w pomieszczeniach w pomieszczeniach pracy-ocena narażania* Bezpieczeństwo Pracy 6/2004

[2] FRONTCZAK M., WARGOCKI P. *Analiza zużycia energii na potrzeby wentylacji i klimatyzacji dzięki zastosowaniu materiałów budowlanych i wyposażenia o niskiej emisji zanieczyszczeń powietrza*

[3] MALICKA D. *Technologia Oczyszczania powietrza ActivTek,* Wentylacja i Klimatyzacja 7-8/2010

[4] CHOCIAJ M. *Jakość powietrza wewnętrznego w świetle Polskich i międzynarodowych uregulowań prawnych* Ciepłownictwo, Ogrzewnictwo, Wentylacja 3/2006

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Factors Affecting Unfreezable Water Content of Bentonites

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Abstract. In this study the effects of cyclic freezing and moisture content on unfreezable water content were investigated. Six homoionic forms of bentonite were used in experimental program. Samples were tested using differential scanning calorimetry (DSC) method. The unfrozen water contents were measurements a method based on DSC method. The method consists in determining the real heat flux function g(T) absorbed by the frozen soil sample during the warming DSC run. It is based on searching for a distribution of ,,heat impulses'' in relation to temperature , which convoluted with the apparatus function a(T) that gives a minimal deviation from the observed heat flux function h(T). The function g(T) can be easily related to the function of unfrozen water content. The analysis of the results was conducted using Universal Analysis 2000 software and JETHRO program. On the basis of statistical analysis it can be assumed that the unfreezable water content of bentonite depends to some extent on the soil moisture content. Influence of cyclic freezing on unfreezable water content was not statistically significant.

Keywords: Bentonite, freeze-thaw cycle, differential scanning calorimetry, unfrozen water, unfreezable water.

1. Introduction

Bentonite belongs to cohesive soils. They are built above all from montmorillonite. This mineral becomes as a result of chemical weathering tuffs in the alkaline and strongly alkaline environment [1]. Montmorillonite is the second after hydromica widely spread clay mineral. The demand for this raw material grows each year, peculiarly in widely understood environmental protection, civil engineering and even medicine. Bentonite structure can undergo modifications as a result of such factors as: soil moisture, cyclic freezing and type of exchangeable cation. Changes of these parameters are conditioning a lot of engineering-geological properties of ground. Influence of exchangeable cation on unfrozen water (the quantity of which depends on temperature) is well know [2]. As the purpose of this work was chosen to determine the effect of total water content and cyclic freezing on unfreezable water which the content doesn't depend on temperature. Water is a factor having strongly influence on bentonite. According to Lebedev [3] we have 5 types of water in the soil: in the form of steam, bound (strongly and weakly tied), free (capillary and gravitational), water in the solid state, crystallization and chemically bound. As knows the temperature of freezing point balance of free water (at constant pressure conditions) is constant and amounts 273.15 K (0°C). In case of soil, value of this temperature (T_0) depends above all on the soil moisture [2, 4, 5]. This temperature constitutes the important parameter in the soil- water system as temperature above which ice is absent in the arrangement. Kozłowski [2] stated that certain part of water doesn't freeze in the temperature T_o. Only further lowering temperature conducts to freezing next portions of water. In moist soil a fraction of water exists, which doesn't change the state of aggregation in a wide range of low temperatures. It is so-called unfrozen water. Its content is usually determined by analogy to the total water content in percent of dry-soil mass (1) , where: m_u – mass of unfrozen water, m_{s} - mass of dry-soil

$$u = \frac{m_u}{m_s} . \qquad (1)$$

In natural conditions the certain part of unfrozen water generally doesn't undergo freezing. This part of water is determined as unfreezable water. The content of the nonfreezable water can be computed as the difference between the total water content and the ice content, which, after assuming , can be expressed as (2), where: L- is the latent heat of melting of ice, in J/g, Δh- is the total heat of phase transition , m_s - mass of dry soil

$$u_n = w - \frac{100\Delta h}{Lm_s} \quad (2)$$

Non-freezable water content in bentonite have some extent connection with strongly tied water so-called hygroscopic. The properties of this water are similar to the property of the solids. This water additionally has a significant viscosity and elasticity. The temperature of its freezing is average -78°C, in addition is dependent on mineral composition. It is estimated that in temperature -70°C still exists 7% of unfrozen water [6], according to this author only at the temperature -193,8°C water in soil completely freezes. Essential here is issue of undertaking the cyclic freezing – thawing to unfreezable water content. Engineering experience convinces us about negative consequences of cyclic freezing and thawing for the subsoil [7]. Additionally in the study of Kozłowski et al. [8] proved that this phenomenon modifies the total specific surface and pore size distribution. What's behind it, can effect of unfreezable water content. According to Anderson and Hoekstry [9], interlayer water in bentonite migrates during freezing to the pore space. In this time the crystallization is held. However, the results of conducted observation scanning electron microscope (SEM) [8] showed that direction of changes the microstructure isn't unequivocal, what additionally make difficult determine the influence of cyclic freezing on unfreezable water content. About diversity and ambiguity of cyclic freezing-thawing processes also wrote Yong et al. [10] and Kumor [7]. This work is an attempt to answer on the question: what extent total water content and cyclic freezing influence on unfreezable water content of monoionic bentonite. As a basic experimental method was used differential scanning calorimetric technique (DSC). Obtained data from every measurement will be subjected to analysis drawn up by Kozłowski [2]. Achieved results were statistically analyzed using the StatCrunch program enabling data analyses in the web.

2. Materials and Methods

The soil used in the current study was: four homoionic forms of bentonite (Ca2+, Mg2+, Na+, K+) and two natural homoionic bentonite with Wyoming (Na+) and Texas (Ca2+). The four forms had been obtained from natural bentonite from Chmielnik in Poland by repeated saturation of the fraction less than 0.063mm and subsequent purifying from solutes by diffusion. The procedure of of preparing homoionic forms of bentonite is performed according to the Kozłowski study [2]. The total cation exchange capacity C.E.C (in %) is 76 in K+ bentonite and Wyoming bentonite and 96 in Ca2+ and Mg2+ bentonite. Cyclically frozen and thawed were realised thanks to differential scanning calorimetry (DSC) technique. Aluminium sample pans were weighed and filled with the soil pastes, sealed hermetically and weighed again. The masses of the soil samples were approximately 10 mg. A thin layer of the soil paste covered only the bottom of the pan, which ensured a very good heat exchange. The DSC Q200 with Tzero built-in technology was used in the experiments. The samples were cooled with the scanning rate of 2.5°C/min to – 90 ºC and after 5min temperature stabilization the sample was warmed with the scanning rate of 5°C/min to +20 ºC. After 30 minutes stabilization the procedure of freezing and thawing was repeat four times. After the experiment, pinholes were punched in the covers, and the soil moisture of bentonite (w) was determined by draining at 110°C. A total of 30 tests were carried out. The results of DSC was plots with five freeze-thaw cycle. Only the results obtained during warming

DSC run were analysed. Thus, the nonequilibrium phenomena connected with supercooling have been excluded. In this study a method of measurement of unfrozen water content based on differential scanning calorimetry (DSC) was used. The method is able to indicate the real phase change in individual soil sample in detail. During analysis of DSC results an identification of peak position creates the problem (along with the temperature of beginning (T_p) and end of the peak (T_k). The research on transitions phase clean crystalline substances gives sharp peaks. Differently is in case of ice melting in the soil-water system which occurs in a wide range of temperatures and at first gives slight thermal effects. The process of ice melting in soil-water system is not a continuous phase change. In this cause, identification T_p and T_k make difficult. Determining these temperatures made according to Kozłowski assumptions [2], which using the method of smallest squares made the approximation line of base and developed search algorithm T_p and T_k. Within new temperatures, including the change of specific heat capacity sample elements in the transformation phase, effected the numerical construction "liquid" base line between [2]. Peak function corrected with regard to the smooth base line h(T)(real heat flux function) describes (3), where : g(T) quasi-linear equation in the temperature interval [T_p ; T_k]; $a_s(T)$,$b_s(T)$- temperature dependent coefficients

$$h(T_i) = g(T_i) - \overline{g(T_i)} \tag{3}$$

$$\overline{g(T)} = a_s(T)T + b_s(T) \tag{4}$$

Data processing from thermal analysis was performed using Universal Analysis 2000 software, thanks to which single cycles were being singled out from the graph and were exchanged for the digital form. The next step in data processing was work in the Jethro program created by Kozłowski [2] for aims of calculating real heat flux function of the converting into unfreezable water content. This way prepared data was copied into Excel and farther subjected to a statistical analysis using the StatCrunch software.

3. Results

Analysis of the variance (ANOVA) and chi-square test confirmed the significance influence of exchangeable cation on unfreezable water content in the individual bentonite type. Test chi-square carried out showed that the influence of mineral composition on unfreezable water content was significant (p-value=0.05) in four from five examined bentonite.

For the evaluation of cyclical freezing influence, total water content and interaction of these factors on unfreezable water was used multiple linear regression carried out using StatCrunch software.

3.1. Effect of Cyclic Freezing-Thawing on Unfreezable Water Content of Monoionic Bentonites

The decomposition of unfreezable water content in individual thaw cycles is shown in Fig. 1. Results of regression analysis show that cyclical freezing doesn't significantly influence on unfreezable water. P-value is 0.83. Visible effect is similar in the influence simultaneously of the total water content and cyclic freezing-thawing on unfreezable water content.

3.2. Effect of Total Water Content on Unfreezable Water Content of Monoionic Bentonites

The influence of soil moisture on unfreezable water demonstrated significant indications of the model. The p-value was 0.039. So single-linear regression analysis was used for the evaluation influence of water content on unfreezable water in individual cycles. Signs of the water content effect to unfreezable water in case four of five above cycles of thawing were observed. In each of cycles the content of unfrozen water is growing with the soil moisture increase.

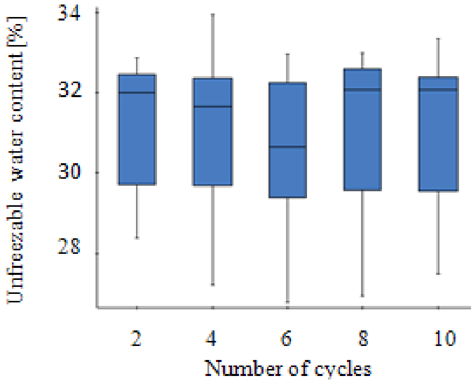

Fig. 1. The unfreezable water content (%) in the various cycles of thawing.

4. Conclusion

Cyclic freezing-thawing practically doesn't influence on the unfreezable water content. It parallels to water adsorbed on flat surfaces of clay particles and probably in the process of cyclic freezing-thawing doesn't change this surface. It is observed certain slight influence of the total water content on unfreezable water content. Possible explanations of this fact are a contribution of the quasi-liquid layer of water on crystals ice to the whole unfrozen water (and as more ice in the sample, the more that type of waters). It is expected that the cyclic freezing will influence on the shape of unfrozen water curve, which the content depends on pore size distribution. However, it requires quite different calculations.

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Author of the article receives a scholarship financed by the European Union under the European Social Fund project "Didactic Potential Development Program at the Kielce University of Technology- education, where success" . The Operational Programme Human Capital, contract number UDA-POKL.04.01.01-00-175/08-00.

References

[1] GRABOWSKA-OLSZEWSKA, B. *Effect of sorption properties of selected types of soils them hydrophilic.* University of Warsaw, 10, 1968

[2] KOZŁOWSKI ,T. *The phase composition of water in clayey soils below 0° C.* Monographs, Studies Dissertations 6, Kielce University of Technology, 1997.

[3] LIEBIEDIEW, A. F. *Soils and groundwater*. Moscow, 1936.

[4] LOW, P. F. *Some themodynamic relationships for soils at or below the freezing point*. Water Resour. Res., 4.

[5] KOZŁOWSKI, T. *Investigation of the freezing point depression in selected model soils*. Arch. Hydrotech., XXXVII (1-2), 199-212.

[6] LITWINOWA, T. A. *Effect of surface and ultra-porosity in frozen ground on the unfrozen water*. Moscow, 1961.

[7] KUMOR, M. K. *Changing the microstructure of monomineral clays and pliocen clay under the influence of cyclic freezing and thawing*. ATR in Bydgoszcz. Dissertations 34, Bydgoszcz 1989.

[8] KOZŁOWSKI, T., GROBELSKA, E., WALASZCZYK, Ł. *Effect of cyclic freezing on microstructure parameters of monoionic bentonites* . University of Zielona Góra, 19, 15-30, 2010.

[9] ANDERSON, D. M., HOEKSTRA, P. *Migration of interlamellar water during freezing and thawing of Wyoming bentonite*. Soil Sc. Soc. Am. Proc., 35.

[10] YONG, R. N. et al. *Alternation of soil behavior after cyclic freezing and thawing*. Proc 4th Int. Sym. Ground freezing, Sapporo, Vol. 1, A. A. Balkema, Rotterdam 1985.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Application of a Laser Diffraction Method for the Determination of the Particle Size of Monoionic Bentonite

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Abstract. In this study, the effect of sample preparation on the content of clay particles was investigated. Monoionic forms of bentonite from Slovakia were used in the experimental project. Samples were tested using laser diffraction. The analysis of the results was conducted using Helos H 2398 SUCEL. Preparation of samples has a significant influence on granulometric distribution of the particles. The most similar results to the areometric analysis were obtained when it was carried out directly after the preparation of samples using the Calgon and sodium carbonate dispersers simultaneously with a mechanical stirrer.

Keywords: Bentonite, laser diffraction method (LDM), particle size distribution (PSD).

1. Introduction

Particle size distribution (PSD) is a significant influencing factor with a lot of physical properties of soil and processes involved with it. It also constitutes the base of general classification of soil according to the PN-86/B-02480 standard. The goal of all particle-sizing techniques is to provide a single number that is indicative of the particle size. However, particles are three-dimensional objects for which at least three parameters (length, width and height) are required in order to provide a complete description. Most sizing techniques therefore assume that the material being measured is spherical and report the particle size as the diameter of the "equivalent sphere" which would give the same response as the particle being measured.

At present there are many methods used to determine the percentage content of individual fractions of soils [1]. The most often applied methods are sieve-pipette method and areometric analysis method. Polish Standard [2] for the evaluation of granulometric composition of soil of the diameter of particles < 63 μm recommends the areometric method. In spite of wide application of the above methods one should remember that they face numerous restrictions, particularly at the measurement of minute particles. Sieve method due to the lack of sieves of the mesh size < 63μm is not suitable for the evaluation of granulometric composition of cohesive soil. On the other hand sedimentary methods are based on Stokes' law, which assumes spherical shape of particles and their identical density [1]. Because of that they do not estimate real dimensions of soil-grown particles. As commonly known [3], particles of clay minerals < 0.005 mm has the form of lamella, rather than spheres. Additionally, the accuracy of readings on the areometr can sometimes make it difficult even a small amounts of organic matter [4]. Thanks to technological advances of several or so years the method more and more often used for the evaluation of granulometric composition is the laser diffraction method (LDM). The technique of laser diffraction is based on the principle that particles passing through a laser beam will scatter light at an angle that is directly related to their size. As the particle size decreases, the observed scattering angle increases logarithmically. The observed scattering intensity is also dependent on particle sizes and diminishes, to a good approximation, in relation to the particle's cross-sectional area. Large particles therefore scatter light at narrow angles with high intensity, whereas small particles scatter at wider angles but with low intensity. This method does not give comparable results with sedimentary methods, because it is based on other principles and conversions but is regarded innovative, anyway. [1, 2]. In laser diffraction, particle size distributions are calculated by comparing a sample's scattering pattern with an appropriate optical model. Traditionally two different models are used: the Fraunhofer Approximation and Mie Theory. In this study laser diffraction was used to examine which rule of operation is based on the Mie model. It can be used to distracted particles with irregular shapes in contrast to the simple model by Fraunhofer, what makes the method more precise. The Fraunhofer approximation was used in early diffraction instruments. It assumes that the particles being measured are opaque and scatter light at narrow angles. As a result, it is only applicable to large particles and will give an incorrect assessment of the fine particle fraction. Application of the laser method for the evaluation of granulometric composition is more universal. Authors of the study [5] think that it best to determine the argillaceous faction with the help of indicating the rate Skemptona. On account of particular properties, they conducted experiments on bentonit. It is argillaceous rock which consists of no less than 75 % montmorillonit. According to scientists, bentonit came into existence as a result of decomposition of ashes and volcanic dusts settled at the bottom of seas in the alkaline environment. It does not usually appear in the pure form. Other clay minerals accompany it, e.g.: kaolinite, illit [6, 7]. These minerals are characterized by a great ability to absorb water which change his structure. Due to strong absorptive properties, these minerals are used in the oil, chemical and food industry. Knowing their composition is an issue of special importance. One should, however, emphasize that apart from the choice of the right research method there exists a sequence of factors affecting results of measurements. One of them is the preparation of the sample for analysis. The paper presents different ways of preparation of samples for granulometric analysis of laser diffraction method. Results were compared to the areometric method, constituting the standard at inspections of classification soils. One should, however, underline, that towards serious simplifying assumptions, areometric method is leaving out „ real " disintegration of fraction of soil and what is more achieved results with laser methods can just match dimensions of real particles more.

2. Materials and Methods

The soil used in the current study was monoionic form of bentonite (Na^+ bentonite). The form had been obtained from natural (Ca^{2+}) bentonite from Jelšovy Potok in Slovakia by repeated saturation of the fraction less than 0.063mm and subsequent purifying from solutes by diffusion. The procedure of preparing monoionic forms of bentonite is performed according to the Kozłowski study [8]. The main physical parameter such as liquid limit is 314% and plastic limit is 47%. The water content is 12%. The grain-size distribution (areometric method) is shown in Fig. 1.

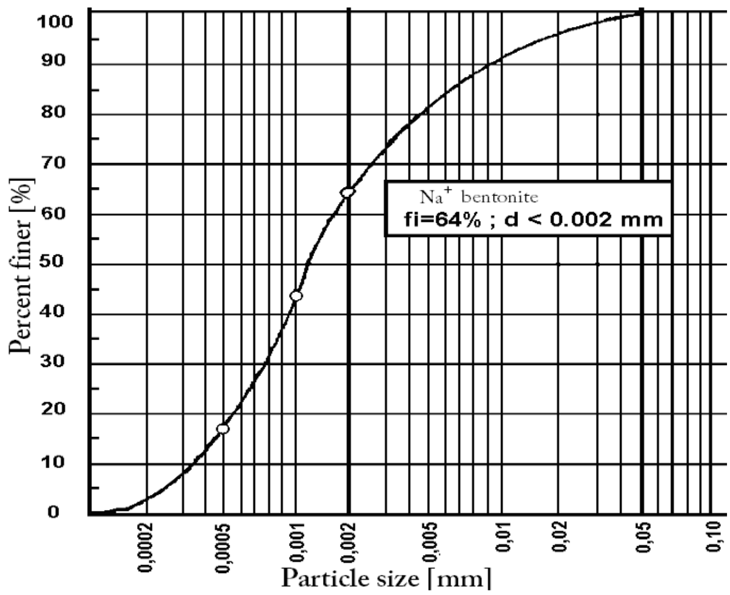

Fig. 1. Grain-size distribution of monoionic (Na^+) form of bentonite.

Particle size distribution (PSD) was measured with laser diffraction method (LDM). HELOS H 2398 SUCELL was used for the LDM. HELOS is designed for determination of grain size distribution of particles within the size range of 0.1- 8750 μm. It makes use of laser lights (blue and red) scattered on measured particles and converts it into PSD. Calculations of PSD were carried out using Mie theory. Mie Theory provides a more rigorous solution for the calculation of particle size distributions from light scattering data. It predicts scattering intensities for all particles, small or large, transparent or opaque. Mie Theory allows for primary scattering from the surface of the particle, with the intensity predicted by the refractive index difference between the particle and the dispersion medium. It also predicts the secondary scattering caused by light refraction within the particle – this is especially important for particles below 50 microns in diameter, as stated in the international standard for laser diffraction measurements (ISO13320-1 (1999)).In this method demineralized water was used as the liquid phase. Before the analysis, the sample was prepared using chemical (Calgon and sodium carbonate) and mechanical (mechanical stirrer) dispersion. In LDM the volumes of the samples were approximately 10ml. Measurements (average of 12 000 images recorder by the detectors) lasted 12 s. Additionally, after 2s stabilization the measurement was repeat two times. Thanks to it, the results was statistically significant. A total of 36 tests were carried out. The results of LDM was grain-size curve.

3. Experimental Tests and Results

Some of the tests was conducted using the sample prepared 48h earlier. The dried soil , at that time, was flooded with distilled water to achieve a plastic consistency(called in this study as paste). Experimental tests are shown in Tab. 1. The same tests was repeated after 48h. Dispersant used in this study was mixture: sodium hexametaphosphate (Calgon-35,7g) and sodium carbonate (7g) dissolved in 1 dm^3 distilled water.

Paste [g]	Dried soil [g]	Distilled water [cm^3]	Dispersion [cm^3]	Mixing		Consistency		Sample prepared and tested		Results [%]		Results after 48 h [%]	
				mechanical stirrer [s]	hand mixing	liquid	slightly-sticky	the same day	The next day	f_i	f_Π	f_i	f_Π
12		200	5	300		x		x		40,2	59,8	31,7	68,3
	10	200	5	300		x		x		36,1	63,9	29,5	70,5
	10	200	20	300		x		x		34	66	27,7	72,3
10		200	5		x	x			x			28,1	71,9
	10	200	10		x	x			x			30,9	69,1
5		5	1		x		x	x		33,3	66,7	29,6	70,4
	4,5	5	1,5		x		x	x		31,9	68,1	28,8	71,2

Observations: The tests after 48 h gives less amount of clay fraction

Tab. 1. The effect of sample preparation (Na+ bentonite) on the content of clay particles.

4. Conclusion

1. Preparation of samples has the significant effect on findings of granulometric composition.

2. In every case the content of clay particles determined by the laser method was lower than the content determined by the areometric method.

3. The result, in the fluid consistency, with the mechanical stirrer, was the most similar to the areometric method for the prepared and inspected sample the same day.

4. The research requires continuation, because apart from the proper preparation of the sample a sequence of factors affecting for precise determining granulometric composition still exists both with laser method (e.g. time of the exhibition for action of ultrasound) as well as areometric (e.g. correct accepting the appropriate density).

5. Desire to gain the maximum similarity of results between laser methods and areometric methods has the practical significance, since the second method is still being regarded as the standard for classification of soil. However, it is possible that due to known simplifying assumptions applied at the areometric method, it is the results obtained by the laser method to better reflect the real particle size distribution in the soil.

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Authors of the article receive a scholarship financed by the European Union under the European Social Fund project "Didactic Potential Development Program at the Kielce University of Technology- education, where success" . The Operational Programme Human Capital, contract number UDA-POKL.04.01.01-00-175/08-00.

References

[1] RYŻAK, M., WALCZAK, R.T., NIEWCZAS, J. *Comparison of soil particle size distribution of laser diffraction method and the method of sedimentation*. Acta Agrophysica, 4(2), 509-518, 2004.

[2] PN-88/B-04481 Building soil. The study soil samples.

[3] KORABIEWSKI, B. *Materials for second-year student of Geography*. Laboratory exercises of soil science.

[4] MYŚLIŃSKA, E. *Laboratory testing of soil*. Warsaw 1992.

[5] CICHY, W., BRYK, J. *Grouting of land by natural bentonite.*, Seminar Materials Geotechnical Aspects of Soil. Gdańsk 1994.

[6] GRABOWSKA-OLSZEWSKA, B. *Effect of sorption properties of selected types of soils on their hydrophilic*. Biul. Geol. UW, 10, 1968.

[7] GRABOWSKA-OLSZEWSKA, B., SIERGIEJEW , J.M. Soilknowing. Warsaw 1977.

[8] KOZŁOWSKI, T. *Phase composition of water in cohesive ground below 0^0C.* Kielce University of Technology, Kielce 1997.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Stiffness of Unbound Base Course of Road Pavement Subject to Mine-Induced Land Surface Deformations

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Abstract. Road pavements built in the area influenced by mining works are subjected to additional loads leading to the reduction of the stiffness of layers. The tensile horizontal strain is believed to be the most dangerous for the functioning of road surface. The conducted research provided an estimation of layer stiffness changes as a result of mining works. Moduli of layers were defined by back-analysis. The tests were carried out on the testing ground (public road) in natural scale along with the whole period of subsidence trough formation. In the successive testing cycles the measurements of road pavement deflection were carried out by using a falling weight deflectometer FWD, the geodesic survey of coordinates XYZ. What is more, the inventory of the road pavement was compiled.

Keywords: Road pavement, ground deformation, mining area, mining induced subsidence trough.

1. Introduction

Circular roads on the site of underground mining are subjected to additional load in the form of horizontal strain (Fig. 1). In course of subsidence trough formation, the horizontal strains develop in the near-surface layer of the rock mass. The strains take on the character of tensions (loosening strains) and compressions (compaction strains) [4][5][6]. The terrain which is deforming due to mining is classified into one of the six categories of the mining area. The categories range from "0" to "V", where "V" is assigned to the major deformations of the terrain.

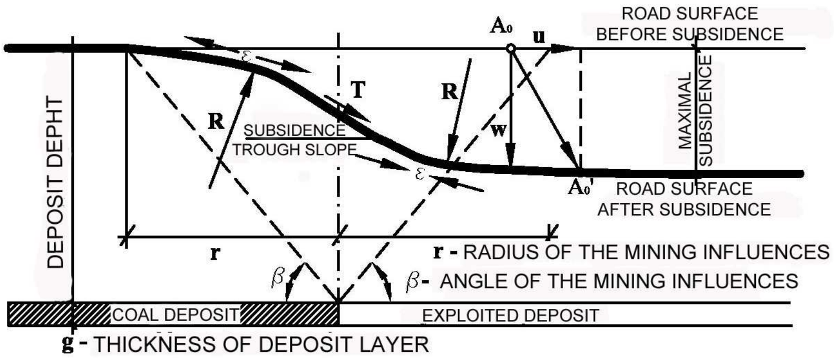

Fig. 1. Mining induced subsidence trough: w – vertical displacement [m], T – inclination [mm/m], R – radius of curvature [km], ε – horizontal strain [mm/m], u – horizontal displacement [m].

The loosening horizontal strains appear in the sub-soil and they affect subsequent layers of the road pavement further on. The following paper presents the influence of loosening strains (ε) on unbound layers. With this end in view, both the measurements with the aid of FWD apparatus (road pavement deflections) and geodesic survey (defining horizontal strains) were carried out.

2. The Characteristics of the Research Section

The research was carried out on a functional highway which is located in the area of deep mining influences – the exploited wall was running in the direction comparable to a vertical one against the centre line (Fig. 2). The mining was carried out at a depth of 810 m, including filling the post-mining void as so-called caving. The road pavement was made in 2002. Inventoried layers configuration of the road pavement construction is presented in Table 1. In course of geotechnical research ground water level was not found to a depth of circa 2,5 m from the road pavement surface. Cracking's failures were not inventoried on the road pavement before the observations [1].

		The deflection measurement point						
Layer	Ordinal	**46**	**47**	**48**	**49**	**50**	**51**	**52**
	km	188,555	188,605	188,656	188,704	188,749	188,803	188,855
Asphalt pavement [cm]	E1	22	16				22	
Aggregate [cm]	E2	68	59				68	
Dusty clay	Ep	> 310	> 310				> 310	

Tab. 1. Inventoried thickness values of the road pavement layers.

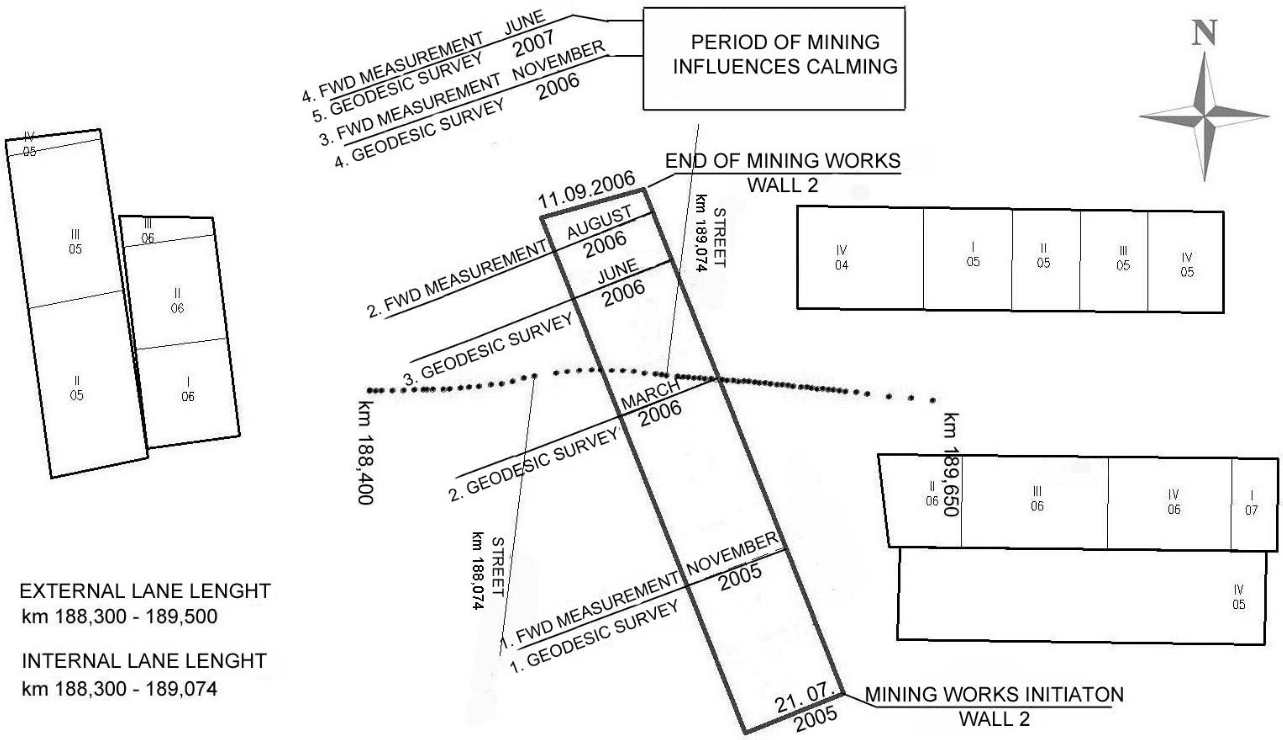

Fig. 2. Location of the mined longwalls with respect to the observed road section [1].

3. Data Analysis

The measurements of pavement deflection were carried out every 25 metres alternately on both traffic lanes. It was done by using FWD apparatus, in different phases of the mining frontage, i.e. before the deformation of the ground surface, during intensive deformations and after the deformations ceased to exist (Fig. 2). Four geodesic surveys were also carried out. As a result, coordinates X, Y, Z were determined on the points stabilized in the road pavement.

Elasticity moduli of the unbound pavement layers were determined on the basis of converse calculations, which use the displacement values obtained from FWD measurements [3]. Calculations were made using iterative method in the Bisar 3.0 programme. Approximation of the displacements measured by the displacements calculated in the model was conducted till the moment when the minimal matching error was obtained. Moduli calculations in a given measuring point were based on revealing moduli values of the first measurement. Searching for the moduli values in the second measurement began with the values obtained from the previous measurement. Moduli of the mineral-asphalt layer (subject to temperature change) and sub-soil (subject to humidity change) were altered in the first place. When the approximation error was high, the modulus of the aggregate layer was also changed. The identification of the moduli from the other FWD measurements was made analogically. The results obtained from the converse calculations were presented with reference to the observed horizontal strains.

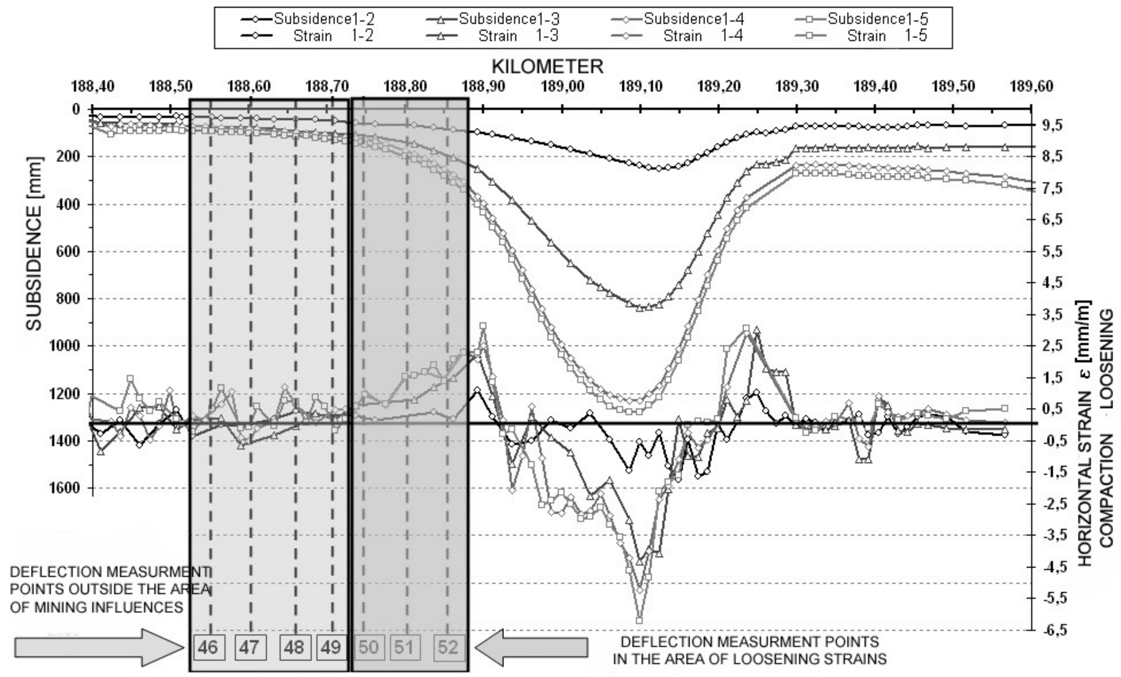

Fig. 3. Estimated indexes of ground deformation [1].

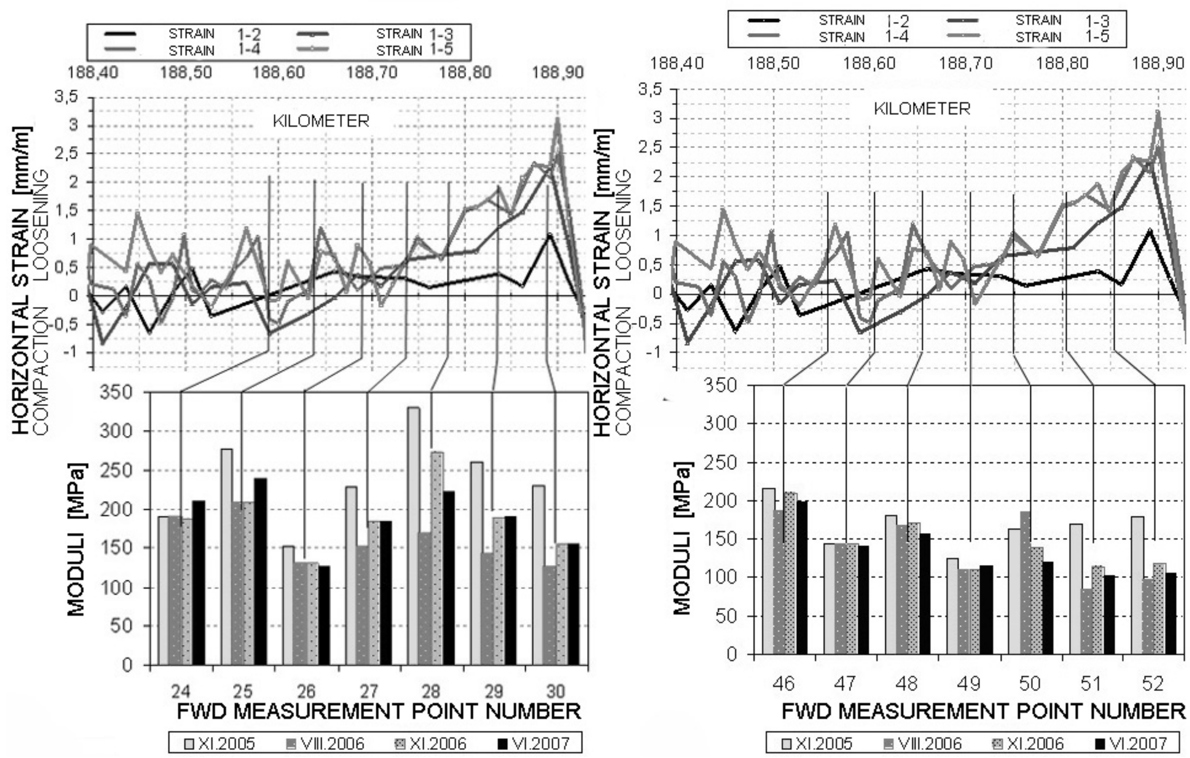

Fig. 4. Identified elasticity moduli of unbound road pavement layers and the observed horizontal strains.

As a result of mining an incomplete trough was formed. Its maximal subsidence was wmax = 1230 mm. On the basis of calculated horizontal strains ε[mm/m], the zones of loosening and thickening strains were appointed (Fig. 3). In the area of analysis (km 188,555 – 188,855) there were constant mining deformations. Section in the 188,750 ÷ 188,855 km was in the area of the major influences. It was subject to horizontal loosening strains which had the extreme value of 3 mm/m. The calculated horizontal strains situate the section in question in the second category of the mining area (Table 1). The area between 188,400 – 188,555 km was situated out of the major influences.

The results of the converse calculations indicated a significant reduction of moduli in the area of loosening strains. In the aggregate layer on both lanes there was about 40÷45% reduction of moduli values (points 28, 29, 30 and 51, 52). Such a significant reduction appeared between the initial measurement and the measurement carried out in the time of intensive loosening deformations. Such a significant reduction of moduli was not recorded in the section outside the major influences ε≤0,5mm/m.

After the period of intensive terrain deformations, there was a partial reconstruction of the stiffness of aggregate layers. The reconstruction was about 7%.

Similar conclusions were drawn for the sub-soil.

4. Conclusions

The influence of continuous mining deformations on the road pavement was being estimated.

The observed horizontal strains did not go beyond the second category of the mining area.

In the area of mining loosening, the decrease (about 45%) of moduli basecourse value takes place.

The observed reconsolidation of the aggregate has a significant influence on the load capacity of the road pavement after finishing the mining. It also requires further investigation.

References

[1] GRYGIEREK M. Wpływ kształtującej się niecki górniczej na trwałość zmęczeniową nawierzchni drogowej. Rozprawa doktorska. Gliwice 2009.

[2] GRYGIEREK M. *Wpływ górniczych rozluźnień podłoża na nośność nawierzchni drogowej w świetle badań terenowych*. Białystok LIV Konferencja naukowa KILiW PAN i KN PZITB T.V, Wydawnictwo Politechniki Białostockiej, Białystok 2009.

[3] SZYDŁO A. *Statyczna identyfikacja parametrów modeli nawierzchni lotniskowych*. Oficyna Wydawnicza Politechniki Wrocławskiej, Wrocław 1995.

[4] KŁOSEK K. Wpływ odkształceń podłoża górniczego na współpracę podtorza nawierzchni dróg kolejowych. Zeszyty Naukowe Politechniki Śląskiej z.68/1988, Gliwice 1988.

[5] CHLIPALSKI K. *Zagadnienia współdziałania nawierzchni drogowej z poziomo deformującym się podłożem górniczym.* Ogólnopolska Konferencja Naukowo – Techniczna Problemy budownictwa drogowego na terenach górniczych, Główny Instytut Górnictwa, Katowice 1997.

[6] CHLIPALSKI K. *Wpływ eksploatacji górniczej na nawierzchnię autostrad oraz ich zabezpieczenie*. Konferencja: Autostrady na terenach górniczych, Główny Instytut Górnictwa, Katowice 1998.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

The Influence of Natural Asphalt Gilsonite Addition on Petroleum Road Bitumen 50/70

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Abstract. The research and analysis presented in this paper is to assess the effect of the addition of natural asphalt Gilsonite at 5%, 10% and 15% on changes a petroleum bitumen 50/70 properties. The research area includes the study: penetration at 25°C, softening point "Ring and Ball", Fraass breaking point and dynamic viscosity at 60°C and 135°C. The results showed a beneficial effect of the addition of natural asphalt Gilsonite on the properties of bitumen 50/70. The use of natural asphalt resulted in a reduction of penetration grade and increasing of softening point, at the same time a small change in breaking point of asphalt 50/70. Tests also showed an effective increase of the dynamic viscosity in operate temperature of 60°C, as well as temperature of 135°C.
Keywords: Gilsonite, asphalt, bitumen 50/70, road.

1. Introduction

The increasing amount of traffic, increasing of axle load cars and more demands from road users in Europe and the world, sped up the development of new roads technologies and has resulted appear of modified bitumen technology.

The aim is to improve the modified asphalt performance of bituminous mixtures and life extension of the road pavement. This requires an increase in surface deformation resistance, fracture, fatigue, aging and external factors [2]. One of the modifying additives of the bitumen, which is achieved through a mixture of asphalt with improved performance characteristics, especially the increased resistance to permanent deformation, is Gilsonite [1].

Gilsonite is a natural, solid hydrocarbons extracted in the mines in the east of the state of Utah, USA. The appearance resembles coal or hard rock asphalt [6].

The production of this unique material began in 1885 when Samuel H. Gilson described the ore discovered in 1860 and named it Gilsonite. Currently, this unique mineral is used in more than 100 products, primarily in the dark paints and varnishes, toners and printing inks, drilling fluids, as a modifier for asphalt and the addition of a wide range of chemical products.

Gilsonite material is brittle and lightweight, crushed a variety of industrially and used as an additive to bitumen form in granulation 0/2mm or in powder form. It occurs naturally in very clean condition, and because it allows him to direct dosing tank or a mixer with a bituminous mix. After dissolving the asphalt is a durable, stable solution does not undergo dissection.

This raw material in its chemical composition contains: Carbon - 84.9%, Hydrogen - 10.0%, Nitrogen - 3.3%, Sulfur - 0.3%, Oxygen - 1.4% and trace elements - 0.1%. The high content of nitrogen, an element that is chemically bound to the amide and amino groups, causes the formation of a kind of anchor to bind to the surface of the polar environment of asphalt mixture, which is more beneficial effect than, for example, use large amounts of paraffins [4]. Due to the low sulfur content Gilsonite not cause environmental pollution. It is not-carcinogenic. Large molecular weight - about 3000, which is 10 times higher than conventional asphalt products from crude oil refineries, it makes Gilsonite to behave similarly to polymers - mass of petroleum asphalt hardening while strengthening and significantly increasing their viscosity.

These advantages of Gilsonite as a modifier for bitumen have been confirmed in practical applications in other European countries including Austria, Belgium, Finland and Slovenia. It was applied to several sections of road. In Poland, this material is still not completely understood, therefore, in this paper was carried out experiment which concerned a influence Gilsonite on bitumen 50/70 properties.

2. Methodology and Test Results

The research, as a input material, was used a petroleum road bitumen 50/70. Modifier - natural asphalt Gilsonite, which appears in the form of powder, was added in an amount of 5%, 10% and 15%. In this way the following samples were prepared:

- asphalt 50/70 with 5% of the natural asphalt Gilsonite,
- asphalt 50/70 with 10% of the natural asphalt Gilsonite,
- asphalt 50/70 with 15% of the natural asphalt Gilsonite.

To determine the effect of the addition of natural asphalt Gilsonite on properties of petroleum road bitumen 50/70 the following research were performed:

- penetration at 25°C, in accordance with PN-EN 1426,
- softening point "Ring and Ball", in accordance with PN-EN 1427,
- Fraass breaking point, in accordance with BS EN 12593,
- dynamic viscosity at temperature 60°C and 135°C.

Changes in the basic properties of asphalt, i.e. penetration and softening point depending on the amount of added modifier are shown in Figures 1 and 2.

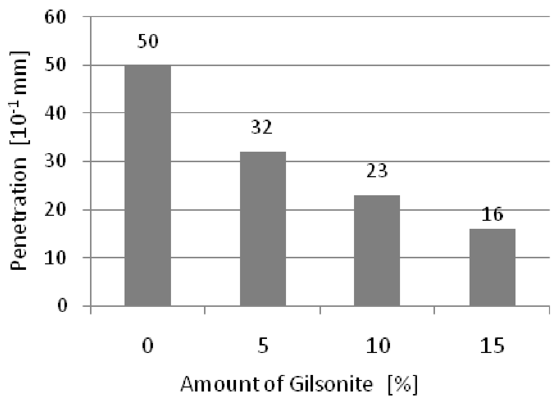

Fig. 1. Penetration value at 25°C versus Gilsonite content.

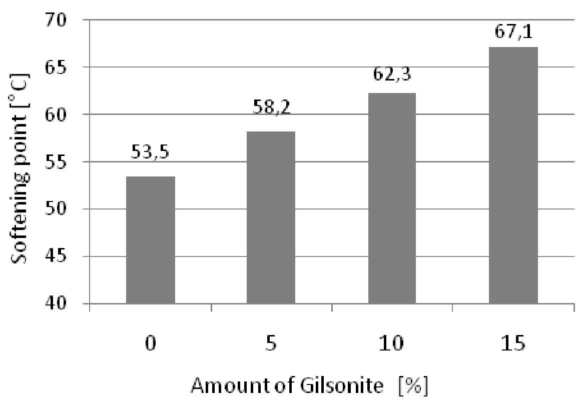

Fig. 2. Softening point „Ring and Ball" value versus Gilsonite content.

The analysis results indicate that the natural asphalt Gilsonite, with an increase in its quantity, affect the changes of selected rheological characteristics of bitumen. Addition of modifier causes significant reduction in the penetration (from $50 \cdot 10^{-1}$mm - bitumen 50/70 to $16 \cdot 10^{-1}$mm - bitumen 50/70 + 15% added Gilsonite) and a proportional increase in softening point. This means that bituminous pavement made of asphalt mix with Gilsonite less susceptible to deformation of the pavement at high temperatures, in comparison with other additives, for example natural asphalt Trinidad Epuré [3].

The results of determination by means of Fraass breaking point shown in Figure 3.
As a breaking point temperature increased as level of Gilsonite get higher. However, this is just a small change in temperature, so you can consider that Gilsonite contributes to crack fracture surfaces at low temperatures.

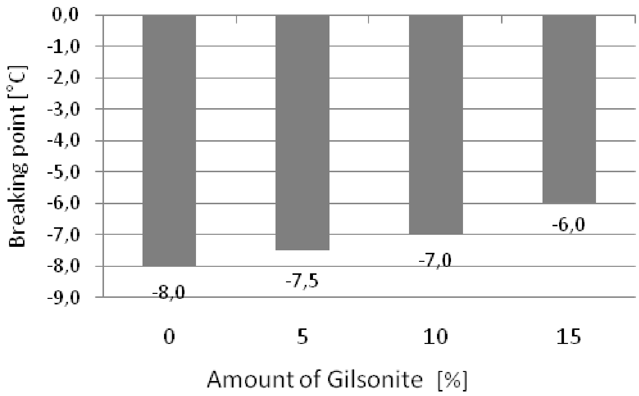

Fig. 3. Fraass breaking point versus a Gilsonite dosage.

Based on the test results of penetration at 25°C, softening point and breaking point, the penetration index PI and temperature range of plasticity TZP were calculated by using formulas [5,6]:

$$PI = \frac{20 \cdot T_{PIK} + 500 \cdot \lg P - 1952}{T_{PIK} - 50 \cdot \lg P + 120}, \qquad (1)$$

where:

T_{PIK} – softening point, °C,

P – penetration grade at 25°C, 0,1mm.

$$TZP = T_{PIK} - T_{łam} \; [°C], \qquad (2)$$

where:

T_{PIK} – softening point „Ring and Ball”, °C,

$T_{łam}$ – Fraass breaking point , °C.

Calculation according to the formula (1) and (2) the penetration index and the temperature range of plasticity of asphalt 50/70 modified with the addition of Gilsonite shown in Figures 4 and 5.

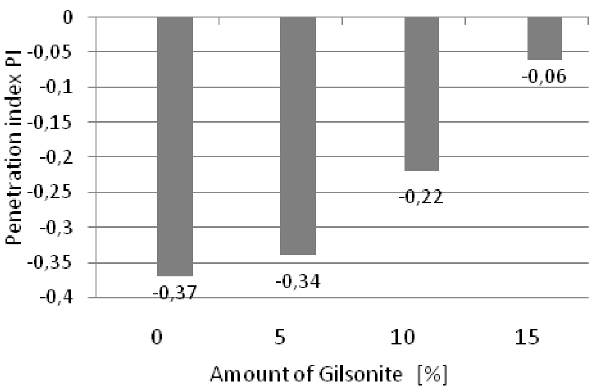

Fig. 4. Penetration index PI versus Gilsonite content.

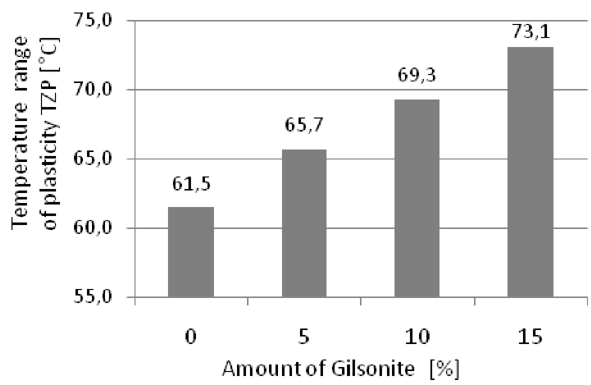

Fig. 5. Temperature range of plasticity TZP versus Gilsonite content.

Analyzing the results obtained from experiments, it should be noted that increasing of Gilsonite content, cause growing of penetration index and reduce the temperature sensitivity of the bitumen. Temperature range of plasticity correlates with the penetration index value, which means that as temperature range of plasticity increases, the penetration index also increases.

An important element of the study was to evaluate the effect of the addition of asphalt Gilsonite on the dynamic viscosity of bitumen 50/70 (Fig. 6 and 7), which is one of the most important parameters of assessment of the bitumen behavior in case of long-term load of road pavement [7].

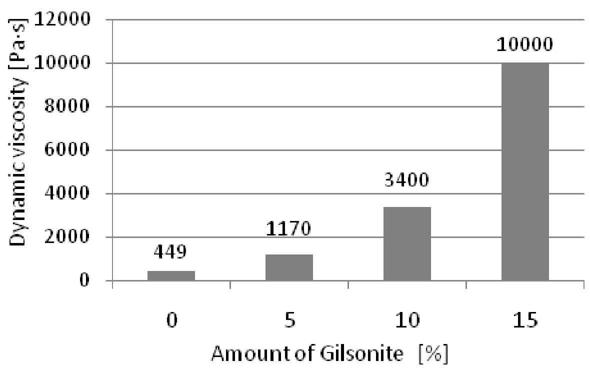

Fig. 6. Dynamic viscosity at 60°C .

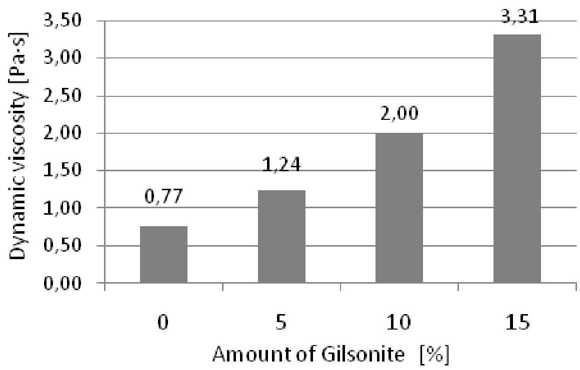

Fig. 7. Dynamic viscosity at 135°C.

Based on the analysis of test results which were shown in Figure 6 and 7 it can be seen considerable increase viscosity in operation temperature of 60°C, as well as at temperature of 135°C. Particularly significant increase was noted at 60°C, where the value of dynamic viscosity of bitumen 50/70 with 15% addition of Gilsonite is 10000 Pa·s and it is over twenty times higher than the dynamic viscosity of the unmodified petroleum bitumen. Therefore, the asphalt which was made on the basis of modified bitumen will be more resistant to the deformations than using a reference unmodified bitumen.

3. Conclusion

On the basis research of road bitumen 50/70 with the addition of natural asphalt Gilsonite the following conclusions can be drawn:

- road bitumen 50/70 modified by natural asphalt Gilsonite was characterized by increasing of a softening point in relation to the road bitumen and lower penetration in each variant of dosage;
- increase in the amount of natural asphalt Gilsonite additive significantly affect the increase of the dynamic viscosity at 60°C, as well as at 135°C;
- modifier additive contributes to the lower thermal sensitivity of bitumen 50/70 and increasing of the penetration index values.

References

[1] BŁAŻEJEWSKI K., STYK S., *Technologie warstw asfaltowych,* WKiŁ, Warszawa 2009.

[2] GAWEŁ J., KALABIŃSKA M., PIŁAT J., *Asfalty drogowe,* WKiŁ, Warszawa 2001.

[3] GRABOWSKI W., SŁOWIK M., BILSKI M., *Ocena wpływu dodatku asfaltu naturalnego Trynidad Epuré na wybrane właściwości asfaltów drogowych,* 56 Konferencja Naukowa, Krynica 2010, s.279-286.

[4] IWAŃSKI M., MAZUREK G., *Odporność na oddziaływanie wody i mrozu betonu asfaltowego z modyfikatorem nisko-wiskozowym,* 56 Konferencja Naukowa, Krynica 2010, s.295-302.

[5] KALABIŃSKA M., PIŁAT J., RADZISZEWSKI P., *Technologie materiałów i nawierzchni drogowych,* Oficyna Wydawnicza Politechniki Warszawskiej, Warszawa 2003.

[6] PIŁAT J., RADZISZEWSKI P., *Nawierzchnie asfaltowe,* WKiŁ, Warszawa 2004.

[7] STEFAŃCZYK B., MIECZKOWSKI P., *Mieszanki mineralno-asfaltowe. Wykonawstwo i badania,* WKiŁ, Warszawa 2008.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Optimization of Foamed Bitumen Cold Recycled Mixes Using Different Binder Contents with Regard to Water and Frost Resistance

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Abstract. The tests were performed on mineral recycled base mixes with foamed bitumen, in which the material from the existing layers was used. The bitumen binder was added to the recycled material in the amount of 2,0% to 3,5%, and hydraulic binder (cement) of 1,0% to 2,5% with changes every 0,5% at variable. The measurements of resistance to the effects of water and frost according to the AASHTO T283 method and the resistance to low temperature cracking according to the PANK 4302 methods confirmed that used foamed bitumen in the cold recycling technology is resistant to these climatic factors. The results obtained were subjected to the optimization process, which allowed to state that with the application of 2,5% foamed bitumen and 2,0% of cement the base course has the required mechanical properties as well as water and frost resistance according to the applied criteria.

Keywords: Foamed bitumen, cold recycling technology, base course, expansion ratio, half-life time.

1. Introduction

Cold in situ recycling using foamed bitumen has gained recognition and popularity around the world as a cost effective technique of rehabilitating road pavement layers. Environmental and economic pressures have caused engineers to reuse the existing pavement materials rather than to import material from quarries. Foaming technology was first introduced by Professor Ladis Csanyi [1] and then further developed by Mobil Oil in the 1960s by creating an expansion chamber. In this laboratory investigation, foamed bitumen was produced by a Wirtgen foaming machine type WLB 10. The. Currently, the pavement cold recycling technology using mineral-cement-emulsion (MCE) mixes is popular in Poland. This bitumen binder enables to obtain more durable road pavements under heavy traffic loads and more unfavourable climatic factors [2].

The cold recycling technology with foamed bitumen is widely used mainly in Africa and Australia, where the impact of temperatures below 0ºC (frost) and water on the road construction is not significant. Consequently, in requirements [3] only tensile strength retained TSR was taken as water resistance criterion in case of this kind of pavement. The climatic conditions in Poland are much more unfavourable with regard to both water and frost interaction with the pavement structure than in countries where this technology is used. In moderate climate conditions no detailed data is available for the pavement structure with foamed bitumen regarding its water and frost resistance. Consequently, it was a vital research aspect to assess the resistance of the material to these two factors in a range which is broader than the requirements prepared for this technology.

2. Foamed Bitumen Tests

In road practice, the bitumen of different penetration is used in the foaming technology worldwide and according to this, an important element of the tests was to preliminarily determine the suitability of bitumen applied in Polish conditions. The tests were performed on six kinds of road bitumen. The suitability analysis covered the determination of the standard and foaming parameters (table 1).

Kind of bitumen	Penetration at 25°C (0,1 mm)	Softening point (°C)	Fraass breaking point (°C)	The optimum water content in the foaming process (%)	Expansion ratio (ER)	Half-life time (τ½) (sec)	Foam Index (FI) (sec)
50/70O	55	50,2	-9	2,5	6,0	8,6	38,9
160/220O	196	41,4	-16	2,5	6,7	11,4	47,3
70/100L	82	47,2	-11	2,5	5,9	6,0	38,6
70/100eL	74	49,4	-12	2,5	6,2	6,3	40,1
50/70N	68	52,8	-10	2,0	10,4	9,7	102,5
80N	81	48,2	-13	2,0	15,1	14,4	185,3

Tab. 1. Characteristics of bitumens used in the foaming process.

Foamed bitumen is characterized by Expansion Ratio (ER), Half-life time (τ½) and Foam Index (FI). The optimum foaming water content was determined by the amount obtain the maximum expansion ratio and the longest half-life time of foamed asphalt. The guidance proposed by Wirtgen [4] is that the minimum permissible values of ER and τ½ should be 8 and 6 seconds respectively. In contrast, CSIR [5] suggest minimum values of 10 and 12 seconds and Asphalt Academy [3] 7 and 7 seconds for ER and τ½. However, according to Jenkins [6] Foam Index should be > 125 second. Using 80N bitumen with 2,0% water content was used to produce the optimum foaming characteristic. Thus, its application for the deep cold recycling should guarantee obtaining pavements of high mechanical parameters.

3. Design of the Mixes

The laboratory tests were carried out on the mineral mix from the rehabilitation construction layers of road surfaces with the addition of foamed bitumen. The mineral mix meeting the grading criteria according to the requirements [3, 7]. The designed mineral mix of the recycled pavement (fig. 2) contained 48% of milled asphalt layers, 22% of the existing stone base and 30% of a new material - 0/4mm dolomite aggregate. In order to determine the influence of the quantity of bitumen blend (foamed bitumen) and the cement (C) on the recycled pavement properties the mix with the content of these two constituents was designed. In tests, as the bitumen binder N80 bitumen was applied, which content in the recycled mix was 2.0%, 2.5%, 3.0%, and 3.5%. In order to determine the influence of hydraulic binder (cement) quantity on the physical and mechanical properties of the recycled pavement its concentration in mixes amounted to 1.0%, 1.5%, 2.0% and 2.5% in relation to the mineral mix.

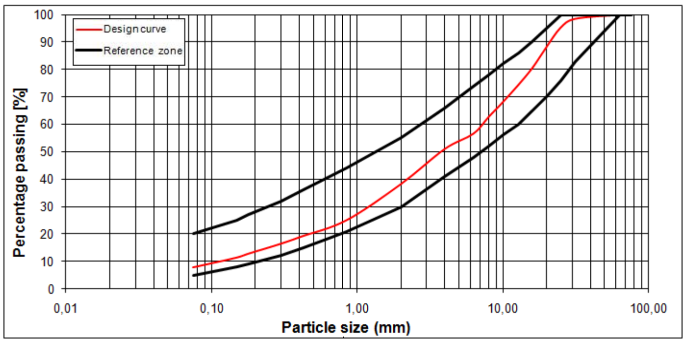

Fig. 1. Grading curves of the mineral mix in the recycling technology with foamed bitumen.

4. Methodology and Analysis of the Test Results

The aim of the research project was to determine the impact of foamed bitumen (FB) and cement (C) content on the mechanical properties of the pavement rehabilitation with the deep cold recycling technology and to determine its water and frost resistance. In order to determine the mechanical properties of the recycled mixes with regard to the amount of binder (FB, C) the following parameters were determined: Marshall stability and Indirect tensile strength (ITSdry and ITSwet). The results of the laboratory tests of the basic mechanical properties have been presented in figure 2.

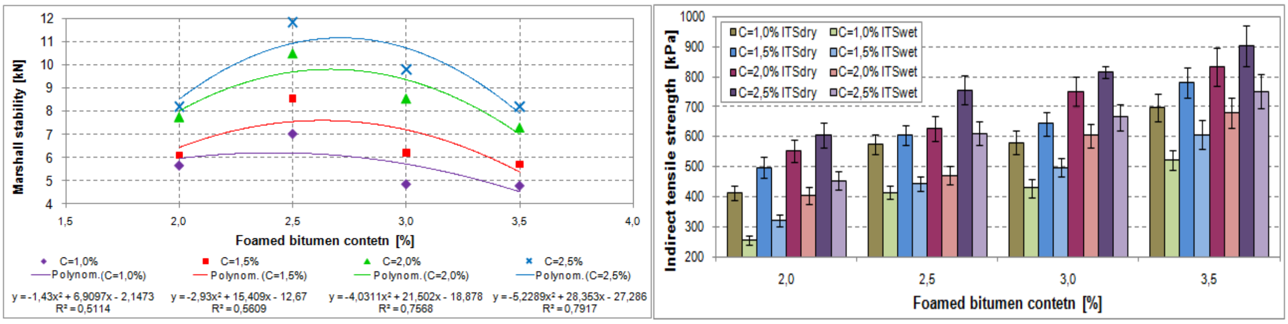

Fig. 2. The impact of the bitumen binder and cement content on Marshall stability and Indirect tensile strength.

The analysis of the tests results presented on the figure 2 of the recycled base mixes reveals that the amount of the applied foamed bitumen as well as the amount of cement have an important impact on the tested features. While assessing the Marshall stability it can be noticed that the mixes with 2.5% FB content have the highest value of the tested parameter and higher than required (8kN) with regard to the technical conditions [7]. Moreover, it was observed that the increase of concentration of FB to 2.5% causes the increase of stability but a further increase causes its decrease. This dependence was not observed in the recycled base ITS test in which, together with the increase of the FB and C contents, the ITS increased. According to the Technical Guideline TG2 [3] all mixes obtain the higher values than the minimal required for this type of mineral material.

In order to assess water and frost resistance of mineral mixes with foamed bitumen the following parameters were determined: tensile strength retained TSR according to the Technical Guideline TG2 [3], resistance to low temperature cracking according to the Finnish PANK 4302 standard [8] and indirect tensile strength after curing in water and frost according to the AASHTO T283 method [9]. The test results of resistance to the effects of destructive climatic factors of the recycled base with foamed bitumen have been presented in figure 3 and 4.

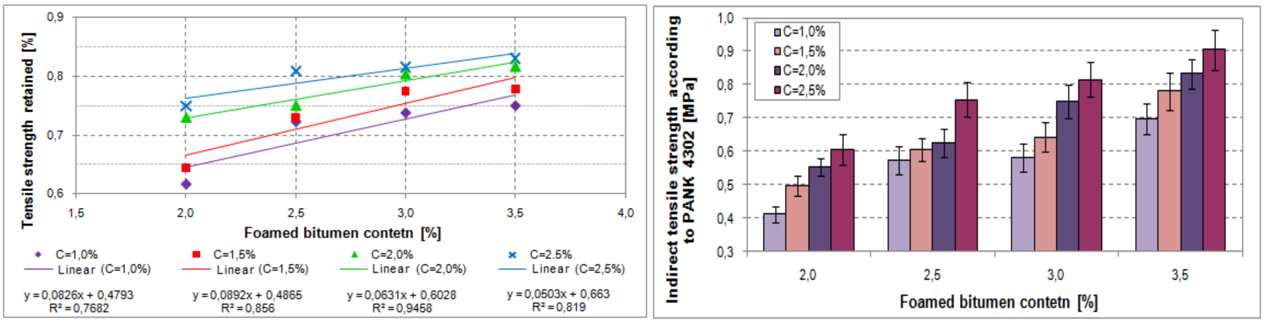

Fig. 3. The impact of the bitumen binder and cement content on TSR and ITS according to PANK 4302.

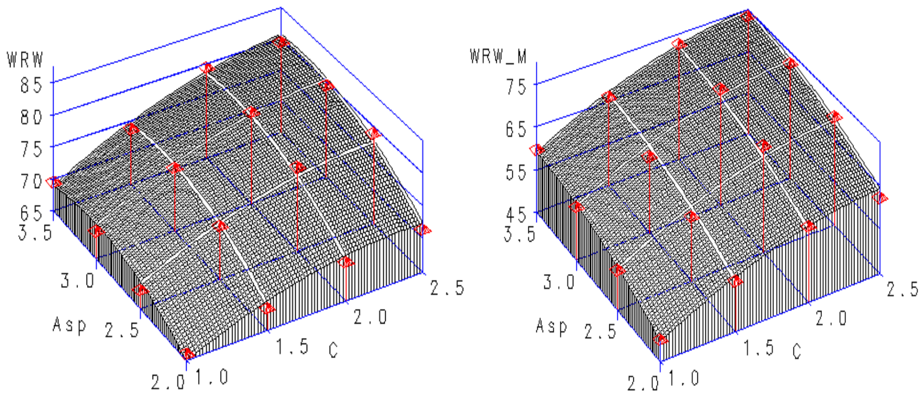

Fig. 4. The impact of foamed bitumen and cement contents on: strength ratio of water resistance WR_W and strength ratio of water and frost resistance WR_{W+M}.

The analysis of test results shows, that the foamed bitumen and cement content impact on the selected mechanical properties of the recycled material. The increase in content of the binding materials in recycled pavement mixes leads to obtaining TSR at the higher level. Tightness and water resistance is ensured with 2.0% cement content in recycled pavement which already contains 2.0% of foamed bitumen. In this case TSR was higher than the required minimal value 0.7. Consequently, it is useless to apply more cement, which can cause cracking in pavement layer and layers of road surface construction above. Indirect tensile strength at -2ºC according to PANK 4302 [8] after the curing process, which simulated the effects of low temperatures, did not exceed the limiting value of 4.8MPa. It can be concluded that as a result, such pavement structures will be resistant to low temperature cracking during winter. A thorough assessment of water and frost resistance of the tested kinds of recycled pavement mixes was performed according to AASHTO T283.

Analyzing the received regression models for tested parameters it is proven that the amount of both binding agents (FB and C) has an important impact on water and frost resistance of the base with foamed bitumen. Together with the increase of bitumen binder and cement concentration the value of the parameters rises. Analyzing the results of the tested pavement mixes for water and frost resistance, it is possible to prove that the required value was not obtained in every case. It should be noted that with 2.5% bitumen binder content and 2.0% cement content the required strength characteristics of tested material were obtained. The strength radio for the tested material was higher than the minimal value equal 70% (WR_W=78.3%, $_{WRW+M}$=70.2%), which assures its water and frost resistance. While higher resistance is ensured by the application of higher content of these two binding materials in the recycled pavement composition.

Basing on the laboratory tests it can be concluded that the water resistance criterion of the recycled pavement basing only on the TSR parameter in Polish climatic conditions should be broadened to included, for example, the proposed AASHTO T283 method.

5. Conclusions

Basing on the analysis of the test results of the recycled base with foamed bitumen the following conclusions can be drawn:

- the ITSdry and ITSwet of base materials with foamed bitumen increases with the increase in foamed bitumen content in recycled pavement material;
- the increase in the foamed bitumen content up to 2.5% in the recycled material for surface pavement improves its Marshall stability; the further increase in the foamed bitumen content causes a deterioration in these parameters;
- the application of 2.5% FB ensures the material’s resistance to the effects of water and frost;
- the foamed bitumen and cement contents of 2.5% and 2.0%, respectively, ensure the required mechanical parameters and water resistance.

References

[1] CSANYI L. H., *Foamed Asphalt in Bituminous Paving Mixes*, Highway Research Board Bulletin, Vol.10, No.160, p. 108-122, 1957.

[2] IWAŃSKI M., *Base with foamed bitumen*, Road Pavement, 3, 97–106, 2006.

[3] Asphalt Academy, *The Design and Use of Foamed Bitumen Treated Materials*, Pretoria, Technical Guideline 2 (TG2), 2002.

[4] Wirtgen GmbH, *Cold Recycling Manual*, 3nd edition published by Wirtgen GmbH, Windhagen, Germany, 2010.

[5] MUTHEN K. M., *Foamed Asphalt Mixes – Mix Design Procedure*, Contract Report No. CR-98/077, SABITA and CSIR Transportek, Pretoria, South Africa, 1999.

[6] JENKINS , K. J., *Mix Design Considerations for Cold and Half-warm Bituminous Mixes with emphasis on Foamed Bitumen*, PhD Dissertation. University of Stellenbosch, South Africa, 2000.

[7] ZAWADZKI, J., MATRAS, J., MECHOWSKI, T., SYBILSKI D. *Technical guideline of performance of the base mineral-cement-emulsion (MCE) mix*, 61, IBiM. Warszawa, 1999.

[8] PANK 4302 - *Standard Asphalt pavements. Low temperature resistance. Method of determining indirect tensile strength.*

[9] AASHTO T283 - *Standard Method for Test for Resistance of Compacted Bituminous Mixture to Moisture Inducted Damage, Standard Specification for Transportation Materials and Methods of Sampling and Testing.*

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Safe Speed Choice from the Viewpoint of Road Users

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Abstract. Speed is at the core of the road safety problem. Strong relationships have been established between speed and both crash risk and crash severity. Drivers often feel inadequate the current speed limit and they might exceed it. Layout of the road should be built in such a way as to induce adequate behaviour and thereby avoid driving errors. The aim of the study was to explore how road users can recognize the various types of roads and what travel speed would they choose in different cases. Result show that, there are some road layouts in Hungary, which are hardly understandable and recognisable for road users.

Keywords: Posted speed limit, preferred speed, safe speed choice, differentiation of road categories.

1. Introduction

Studies revealed that driver related factors were solely to blame for around 50% of accidents on roads. When combinations of driver factors and environment or vehicle are added, the driver was involved in more than 90% of accident causation factors [1].

Speed is at the core of the road safety problem. Strong relationships have been established between speed and both crash risk and crash severity. Even if inappropriate speed is the direct cause of an accident, the specific site obviously provokes this unsafe behaviour [2].

A self-explaining road is a road designed and built in such a way as to induce adequate behaviour and thereby avoid driving error. A perfectly designed self-explaining road would not require speed limit signs or any warning signs [3].

According to main principles of safe road design, the function, design and use of the road should be coherent with each other. To reach the right road usage and to avoid driving errors, layout of the road has to meet with the function of the road [4].

2. Previous Studies

Speed limits are generally meant to provide information to the driver about the speed (s)he can drive safely in average conditions. However, setting a limit does not automatically result in the required speed behaviour. If drivers consider a posted speed limit incredible or inappropriate for a given road section, they may well ignore that limit and form their own decision as to what speed is appropriate. This assumption is affirmed by several survey studies.

2.1. Study about the Credibility of Speed Limits from Netherlands [5]

In 2007 600 Dutch car drivers were asked in a form of questionnaire to judge 27 photographs of (different) rural roads with a posted speed limit of 80 km/h. To determine the degree of credibility, for each road scene the respondents filled in the preferred speed and the speed limit they considered to be safe. The results show large differences in both preferred speed and the safe speed limit between the road scenes, both below and above the limit of 80 km/h. These differences were related to a number of characteristics of the road and the road environment, such as the presence or absence of a curve and characteristics concerning the field of view (sight distance, clarity of situation). Subjects were influenced by more or less the same road features.

2.2. Study about the Attitudes towards Speed Limits in Australia [6]

In Australia an online survey was administered, with a total of 4100 respondents recruited. The survey focused on attitudes towards speed limits for four different road types, and the sample was stratified according to age, gender, and area of residence. It was found that most respondents were able to correctly identify the speed limit for local residential streets and major urban arterials, although their knowledge of rural speed limits was considerably lower.

The majority of respondents were in favour of the proposed lower speed limits on 100 km/h two-lane undivided rural roads and on rural gravel roads, but only about one-third supported lower limits in urban areas. A cluster analysis revealed that there were varying characteristics between respondents who were more or less likely to support speed limit reductions, across a number of demographic, socio-economic status, and driving behaviour variables.

2.3. Study about the Effects of Centreline on the Speed Choice from the United States [7]

In February 2011 Professor Norman W. Garrick made a smaller survey amongst his students during a university lesson at University of Connecticut. Students were divided into two groups. They were shown separately 12 - 12 different road scenes and their task was to choose the speed that they feel adequate and safe at actual situation. One group got road scenes with centerline, the other got the same pictures without centerline, erased with an image editor program.

Taking the means and medians of answers of two matching images, the result shows differences considering other parameters of the road section. Most results show that road with the centerline looks more like a rural highway, as higher speed results proving about, while without it looks more like a residential area; thus, the slower speeds were chosen. The double yellow centerline makes road seem much more like a rural highway. There were also scenes, when respondents seem to be affected more by context and parking cars than by the presence or absence of painting in the middle.

3. Survey

Our study was a questionnaire survey in which students had to judge photographs of road scenes. The participants were asked to state what speed they preferred for each road scene. Participants were not informed about the actual speed limit.

The aim of the study was to explore how road users can recognize the various types of roads. Furthermore we would like to find out if there is any difference between the posted speed limit and the chosen speed and how much this difference is.

3.1. Respondents

The target group of the current survey was a specific group of society. The sample was 215 students, from a university lesson at Széchenyi István University in Hungary, in the city of Gyor. Their average age was 25. Most of them own a driving licence. The average period of driving licence possession was 7 years. 80% of participants were male, 20% female. This sample is not representative of the Hungarian population of license holders in terms of age and gender, but can be a part of our future research.

3.2. The Questionnaire

Participants completed the questionnaire at home at their own computer. The questionnaire consisted of two parts. The first part contained questions on age, gender, driving licence possession and the number of years of licence possession, about the driving practice and style. The second part of the questionnaire consisted of 35 photographs of real life road scenes from Hungarian roads. Respondents got randomized sequence of photographs in order to rule out possible sequence effects.

Photographs depicted motorway scenes, expressway scenes, primary roads with elevated speed limit and rural primary road scenes. Some of them had physical separation between traffic directions, others didn't. The seven categories and their main characteristics are shown in Tab. 1 and in Fig. 1. Each category had 5 photographs in the questionnaire. For each of the 35 photographs the respondent had to fill in the speed that (s)he would like to drive in that situation. From the sample participants were deleted whose average preferred speed differed from the average value at least with the twice of the standard deviation and also those, who didn't own a driving licence.

	Road class	Posted speed limit (km/h)	Number of traffic lanes	Physical separation of directions	Number of road scenes shown
1	motorway	130	2x2 lanes	with p. s.	5
2	primary road with elevated speed limit	110	2x2 lanes	with p.s.	5
3	primary road with elevated speed limit	100	2x2 lanes	with p.s.	5
4	primary road with elevated speed limit	100	2x2 lanes	without p.s.	5
5	expressway	110	2x1 lanes	without p.s.	5
6	primary road with elevated speed limit	110	2x1 lanes	without p.s.	5
7	primary road	90	2x1 lanes	without p.s.	5

Tab. 1. Main characteristics of the road categories surveyed.

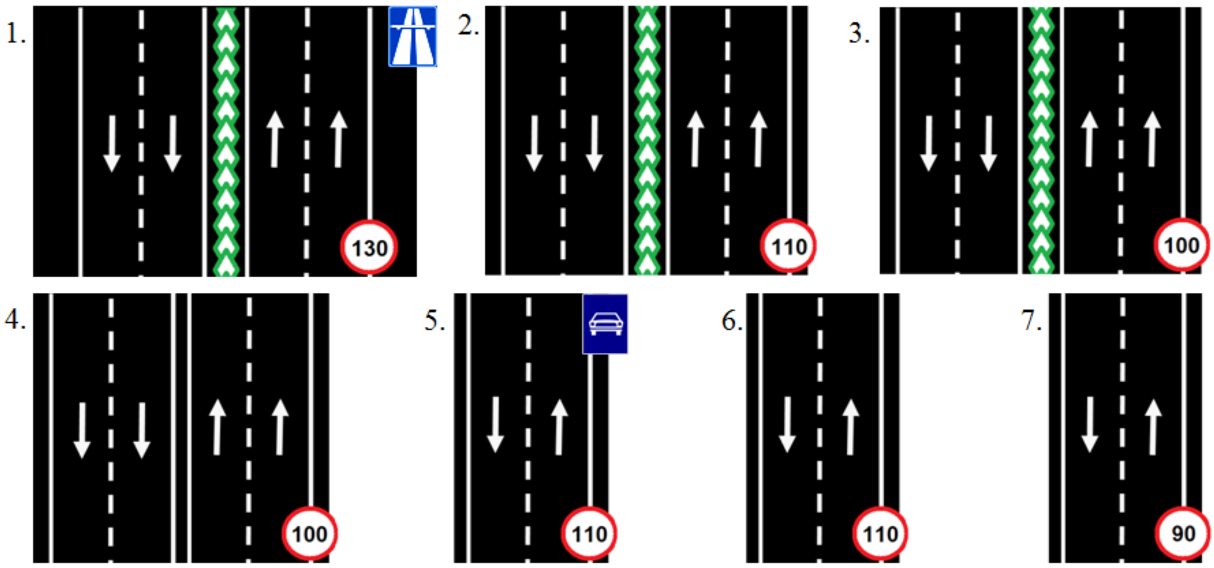

Fig. 1. Schemes of seven road categories and their posted speed limits.

4. Results and Discussion

For each picture, the average preferred speed, the v_{85} speed, the standard deviation and the difference from the posted speed limit was calculated. The average results for the seven categories are shown in Tab. 2.

For motorways (column 1) the average preferred speed is only about 7 km/h lower than the speed limit and speed v_{85} is adequate for this category. The relative standard deviation is only 11%, thus it is clear that people can easily recognise this category. The situation is similar for primary rural roads with 2x1 lanes (column 7).

As for the primary roads with elevated speed limit 110 km/h divided (column 2), the relative standard deviation is 13%, as drivers choose different speeds due to the unfamiliar situation. The v_{85} speed exceeds the posted speed by 11 km/h.

A bit strange are the results in the case of undivided 2x1 lanes roads: on expressways and primary roads with elevated speed limit 110 km/h (column 5 and 6). Participants might perceived these scenes as normal primary rural roads, so the preferred speed is lower by more than 10 km/h from the posted speed limit and also the v_{85} speed does not reach the posted speed limit.

The results are worrying in the rest two cases, on divided 2x2 lanes primary roads with elevated speed limit 100 km/h (column 3) and also on undivided 2x2 lanes primary roads with elevated speed limit 100 km/h (column 4). In these cases the average chosen speed is higher than the posted speed limit and also the v_{85} speed exceeds the posted speed limit by more than 10 km/h, which means that on these roads the driver might feel that, s(he) is passing on a higher road category.

category	1	2	3	4	5	6	7
posted speed limit (km/h)	130	110	100	100	110	110	90
average of answers (km/h)	122.8	107.0	111.4	101.5	98.7	96.2	85.3
difference (km/h)	-7.2	-3.0	+11.4	+1.5	-11.3	-13.8	-4.7
standard deviation (km/h)	13.2	13.9	14.5	11.1	10.7	10.5	8.6
standard deviation (%)	11%	13%	13%	11%	11%	11%	10%
v_{85}(km/h)	136.5	**121.4**	**126.4**	**113.0**	109.8	107.1	94.3

Tab. 2. Results for different categories.

Our further objective is to address other subgroups in the Hungarian population as targets for future road safety research and reach similar results which are generally true for Hungarian drivers and to compare our results with speed measure data and with accident statistics on these roads.

5. Conclusion

The results show that some road categories like motorway and primary rural road are very well understandable for road users and speed choice is clear for them. There are also road categories, which are hardly recognisable for road users. In the case of primary roads with elevated speed limit with 2x2 traffic lanes the road users usually exceed the posted speed limit. The layout of the road might not meet with the function of the road, which leads to inadequate speed choice and can cause safety problems. On expressways and primary rural roads with elevated speed limit, which have 2x1 traffic lanes, road users tend to drive much slower than the posted speed limit, because they don’t feel safe higher speeds at these categories.

Acknowledgement

The preparation of this paper was supported by TAMOP-4.2.1/B-09/1/KONV-2010-0003: Mobility and Environment. The Project is supported by the European Union and co-financed by the European Social Fund.

References

[1] TREAT, J. R., TUMBAS, N. S., MCDONALD, S. T., SHINAR, D., HUME, R. D., MAYER, R. E., et al..: *Tri-level study of the causes of traffic accidents,* Volume I: Casual factor tabulations and assessment. Final report (No. DOT-HS-034-3-534), Washington: National Highway Traffic Safety Administration (1977)

[2] WELLER, G., SCHLAG, B., GATTI, G., JORNA, R., van de LEUR, M.: *Human Factors in Road Design. State of the art and empirical evidence,* Road Infrastructure Safety Protection – Core-Research and Development for Road Safety in Europe; Sixth Framework Programme (2006)

[3] LIPPOLD, Ch.: *Road Design - New Types of Rural Roads in Germany*, Conference MAUT 15 years (Oct. 2009)

[4] KOREN, Cs.: *Sustainable safety,* University lecture, Széchenyi István University, Gyor (Feb. 2011)

[5] GOLDENBELD, Ch., van SCHAGEN, I.: *The credibility of speed limits on 80 km/h rural roads: The effects of road and person(ality) characteristics*, Accident Analysis and Prevention 39 (2007), pp. 1121–1130.

[6] LAHAUSSE, J. A., van NES, N., FILDES, B. N., KEALL, M. D.: *Attitudes towards current and lowered speed limits in Australia,* Accident Analysis and Prevention 42 (2010), pp. 2108-2116.

[7] GARRICK, N. W.: *Speeds and Street Design Results UConn and UCD*, Highway design class, University lecture, University of Connecticut (Feb. 2011)

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Emitted Particulate Matter around Roads

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Abstract. Dustiness represents a big problem in department of air pollution in the surrounding of highways. Dust air pollution is marked mainly in cities with heavy street pattern and heavy traffic. A mass of dust particles includes particles from different sources, with different dimensions, and different chemical composition in the air. Each characteristic of particulate matter – PM predestines their time of abidance in ambient air, their ability of long-distance transport and especially their toxicity for environment – health of population. First of all, the need for protection health of population is determining for urgency detect quantity of particulate matter in the air. Realized measurements are basis of set limiting steps for decrease pollution.

Keywords: Particulate matter, road, road traffic, air pollution, PM10, PM2,5.

1. Particulate Matter – PM

PM includes particles of solid and fluid mass about the size from 1 nm to 100 µm, which stayed in the ambient air. PM is situated like complicated heterogeneous mass, in term of size of particles and chemical composition, in atmosphere. Specific physical (design, size (Fig. 1.), electrical charge, surface particles and solubility) and chemical properties (inorganic and organic parts) depend on them source, mechanism formation and next conditions (distance from source, meteorological conditions). Physical properties - particles fractions are mainly determining for emitted particles. Riskiness of PM doesn´t consist just in mechanical conditions but mainly consists in contents danger organic (PAH – polycyclic aromatic hydrocarbons) or inorganic harmful pollutants (heavy metals) [1].

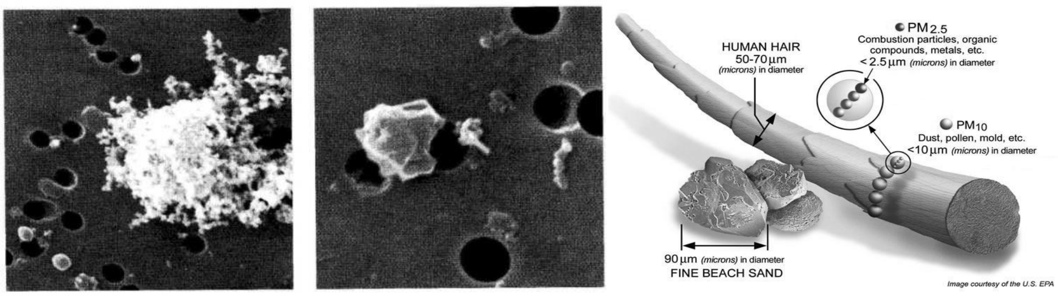


Fig. 1. PM is caused by combustion fuel (zoom 30 000 times) (left), PM is caused by mechanical separation (zoom 70 000 times) (in the center) [1] and comparison particulate matter size with human hair (right) [2].

PM is emitted directly or formed secondarily in the atmosphere. Secondary particles are formed from primary gaseous emissions in the atmosphere. Examples of secondary particles include sulfate, formed from SO2 emissions, and nitrates, formed from NOx emissions. Particles are defined for regulatory purposes in the U.S. as either fine, with an aerodynamic diameter less than 2,5 microns ($PM_{2,5}$), or coarse, with an aerodynamic diameter in the range of 2,5 to 10 microns ($PM_{10-2,5}$).

The chemical composition of particles depends on location, time of year, weather and source.Adverse health effects from exposures to PM result from penetration and deposition of particles in the various regions of the respiratory tract, and the biological response to these particles. Adverse health effects include decreased lung function, exacerbation of asthma symptoms, and changes to lung tissues and structure. Furthermore, in recent years, results from numerous studies have provided evidence of an association between premature mortality and morbidity, and exposure to particulate matter. This association appears to be attributable to fine rather than coarse PM. In addition, source specific factor analysis shows that an increase in mobile source fine PM is associated with increased mortality. The data also indicate an association between long-term exposure to fine PM, cardiopulmonary mortality, and lung cancer [2].

Particulate matter is not just from one source but particles are mixed from different sources – stationary sources or mobile sources in the ambient air. In both cases, particulate matter can come from exhaust or non-exhaust processes.

2. Measurement of Particulate Matter by the Roadsides

Measurements of particulate matter are realized by the roadsides in city of Žilina and once were realized by the highway in Predmier. First object of measurements is long time monitoring proportional representation of particulate matter in the air PM_{10}, $PM_{2,5}$ a $PM_{1,0}$ and their behavior concerning ambient conditions. The chemical analyze of particulate matter will be realized in second phase and also determination possible source of PM.

Measurement location is placed in Žilina on street V. Spanyola near Regional authority of public health (RUVZ) and second location was in Predmier by the highway D1 in areal of Centre management and maintenance of highways (SSUD).

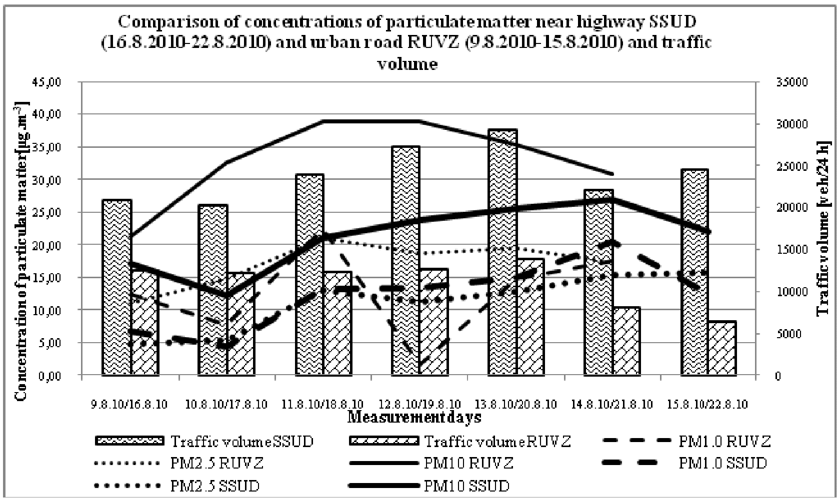
Quantity of particulate matter is assigned with using reference method STN EN 12341 a STN EN 14907.

Low-volume samplers LECKEL LVS3 are used for measurement in number 3 pieces (Fig. 2.). Deviance for statistic of traffic driving SIERZEGA is used for detection traffic volume (Fig. 2.).

Fig. 2. Deviance for statistic of traffic driving Sierzega (left) and low-volume sampler Leckel LVS3 (right).

Particulate matter is produced in diferrent sizeses to the air. Smaller particulate matter = bigger health risks. Monitoring of fractions of particulate matter is very important for urgency to define limits not only for PM_{10} but also for smaller fractions.

Concentration of particulate matter ($\mu g.m^{-3}$) was different on selected locations and it was, in same case, twice lower on location SSUD like on RUVZ (Fig. 3.). Maximum value of fraction PM_{10} 38,81 $\mu g.m^{-3}$ was on location RUVZ and 26,95 $\mu g.m^{-3}$ on location SSUD. Traffic volume is twice higher on location SSUD like on RUVZ (Fig. 3.). Higher concentration of particulate matter can be caused by worse dispersion conditions in the city how around highway on open space, possible by other sources of particulate matter – big stationary sources. The course fraction contained more than 50 % particulates of fine fraction on both locations.



Fíg. 3. Concentrations of particulate matter and traffic volume on measurements locations – August 2010.

Concentration of particulate matter in October 2010 was higher like in August 2010 (maximum value of PM_{10} was 96,87 $\mu g.m^{-3}$) and fine fraction was 70% from caurse fraction at average (Fig. 4.). Influence of road traffic is not so clear on concentration of particulate matter in October 2010 (Fig. 4.). It can be caused by heating season, mainly central or lokal combustion of solid propellant (wood), what can contribute for higher concentration of particulate matter.

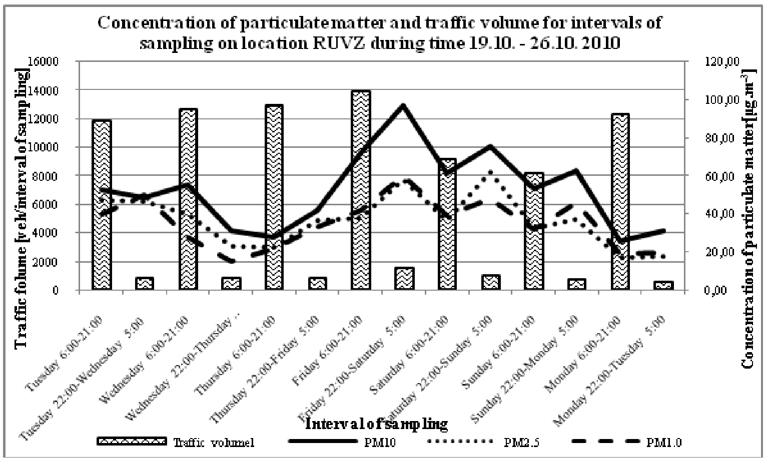


Fig. 4. Concentration of particulate matter and traffic volume on location RUVZ – October 2010.

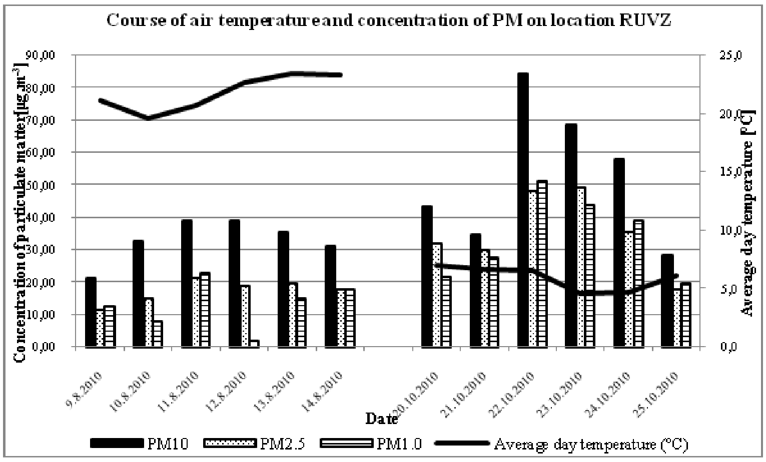


Fig. 5. Course of air temperature and concentration of PM on location RUVZ – August 2010, October2010.

Higher concentration of particulate matter can be caused by light air circulation in colder time, along horizontal or vertikal direction (Fig. 5.).

3. Conclusion

Distribution of particulate matter size is important task for assessment of air quality, therefore what fractions of particulates are in sampling and testing air. We breathe this ambient air and it´s necessary for live. Discover sources of PM is needed in case of the bad quality of air, which participate in enormous situation and draw consequences. Road traffic is one of possible sources of air pollution by particulate matter but not the only. How we could see presentation results, also in higher traffic volume, can be concentration of particulate matter lower or reversely. Higher concentration of particulate matter was discovered on location RUVZ overnight, when the traffic is minimal. Measurements were realized during winter, during heating season. We will be continuing in research studies and in measurements on the same location during summer, what can extend view in air pollution by particulate matter produced by road traffic.

Acknowledgement

This contribution is the result of the project implementation:

Centre of excellence for intelligent transport systems and services, ITMS 26220120028 supported by the Research & Development Operational Programme funded by the ERDF.

"We support research activities in Slovakia/Project is cofinanced from sources of EU"

The paper has been worked out with support from the scientific research task VEGA 1/0508/11 picked up again 1/0673/08.

References

[1] Adamec, V. et. al. 2008. *Doprava, zdraví a životní prostředí*. Praha: Grada Publishing, a. s., 2008. 176 s. ISBN 978-80-247-2156-9.

[2] U. S. Environmental Protection Agency. Particulate matter (PM) research. [online]. [citované 15. 02. 2011]. Dostupné z <http://www.epa.gov/ord/ca/quick-finder/particulate-matter.htm>

[3] Baldauf, R., McDonald, J., Heck, R., Cook, R., Rege, J., Audette, L., Armstrong, J. *Management of motor vehicle emissions in the United States*. In Elsom, D.M., Longhurst, J.W.S.. Regional and local aspects of air quality management. Southampton, Boston: WITpress, 2004, s. 119-155. ISBN 1-85312-952-6.

[4] STN EN 12341: *Ochrana ovzdušia. Určenie frakcie PM10 poletujúcich častíc*. Referenčná metóda a skúšobné postupy v teréne na dôkaz referenčnej rovnocennosti meracích metód.

[5] STN EN 14907: *Ochrana ovzdušia. Vonkajšie ovzdušie*. Štandardná gravimetrická metóda merania na zisťovanie hmotnostnej frakcie PM2,5 suspendovaných častíc

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Experimental Analysis of Fatigue Prone Structural Detail of Riveted Steel Railway Bridges

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Abstract. The paper deals with fatigue resistance of a typical fatigue prone structural detail of riveted steel railway bridges - the connection of stringer to cross-beam. The joint, in which only the webs of stringer and cross-beam are connected, used to be verified only on the shear and normal forces. However, due to the connection arrangement, the normal bending stress at the edge of the stringer web arises and results in creation and development of fatigue crack. The categorization of this typical fatigue structural detail is not clearly defined in EN 1993-1-9. In order to investigate and define the fatigue category of this detail more properly, the laboratory tests on specially adapted specimens were performed. Based on fatigue test results, the fatigue category of this detail was specified.

Keywords: Steel bridges, specimen, fatigue detail, stringer.

1. Introduction

Fatigue resistance of riveted steel bridges, which were built mostly in the first half of 20th century, represents one of the determining factors on decision-making related to next exploitation of them after finishing their planned life time. The standard fatigue assessment method according to Eurocode EN 1993-1-9 [1] is based on the categorization of the fatigue prone structural details. However, the categorization of riveted details in the standard mentioned above is not sufficient for practical fatigue assessment. The paper deals with the fatigue resistance determination of the typical fatigue prone structural detail of riveted steel railway bridges - the connection of stringer to cross-beam [4]. This structural detail is typical by frequent occurrence of fatigue cracks (see Fig. 1), but its categorization according to standard mentioned above is at least questionable at least. In order to investigate and define the fatigue category of this detail more properly, the laboratory tests on specially adapted specimens were performed. Then, experimental results were used to specify the fatigue category of the detail.

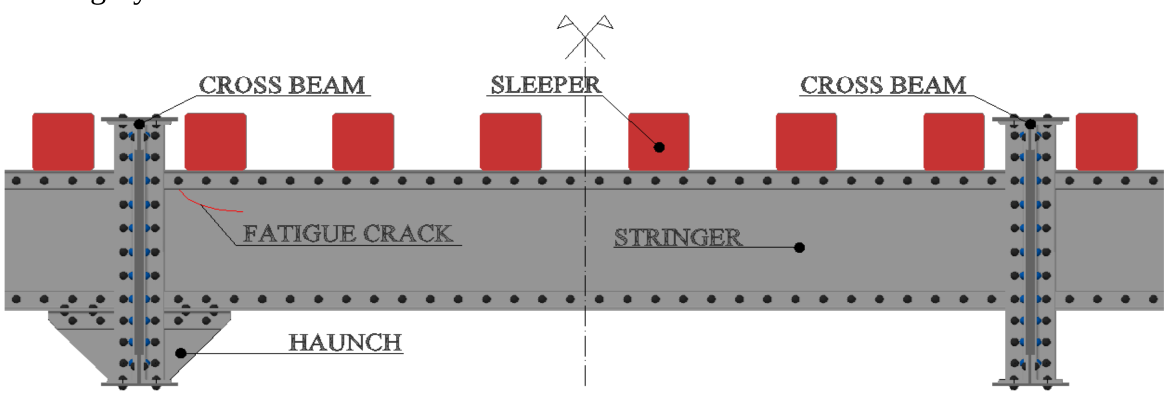

Fig. 1. Arrangement of a typical stringer to cross-beam connection with/without reinforcing haunches (example of fatigue crack).

2. Laboratory Testing Process

2.1. Laboratory Specimens

Because of limited possibilities of the applied pulsating device as well as due to the economic aspect, the common static scheme of real stringers as a simply supported beam (supported by cross-beams), loaded by reactions from sleepers (see Fig. 1), could not be applied. Therefore, a cantilever loaded by one force at the free end was used as the static scheme for laboratory testing (see Fig. 2). Six specimens were manufactured of steel S235 in all. All specimens were consisted of the web from steel plate of P 10 x 390 – 1115 and flanges made of two angels L 80 x 80 x 8 – 995 mm connected to the web by means of rivets with diameters of 22 mm. Stringer was connected to the hot rolled cross-beam of IPE 700 by means of two angels L 80 x 80 x 8 - 692 mm using 16 bolts of M 24 – 8.8 in the case of specimens of type I and II. Three specimens were reinforced by triangular haunches at the connection to cross-beam (Fig. 2b) and other three specimens were manufactured without the haunches (Fig. 2a).

After testing the six aforesaid specimens, another five specimens of type III (see Fig. 2c) were obtained adjusting the previously tested stringers. The connecting angles were cut off and the new connecting angles L 120 x 120 x 12 – 400 mm were riveted to the free end of the specimens by means of 10 bolts of M 24-8.8. The new connecting angles were shortened in comparison with previous six specimens in order to situate measuring gauges above the first and below the last rivet, respectively, since the fatigue crack was supposed to develop from this point. Thus, eleven test specimens were tested altogether.

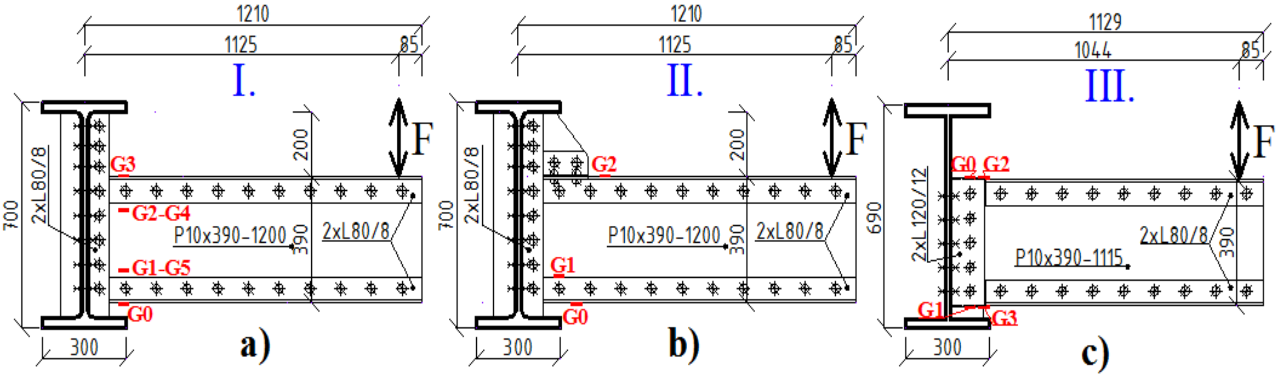

Fig. 2. Configuration of a laboratory specimens without haunch (I), reinforced by haunch (II) and new specimens without haunch (III) and specimen gauge arrangement.

2.2. Testing Process

The specimen types I and II were tested in 2009 and the testing process as well as the test results were described in [2]. Other five new specimens were tested in the laboratory of Transport Research Institute in Zilina from January to April 2010.

All the specimens were gradually subjected to fluctuating bending moment through the application of concentrated vertical load situated at the distance of 1 125 mm from the supporting cross-beam (see Fig. 2) in the case of specimen type of I and II and 1 044 mm, in case of specimen type of III respectively. For all specimens, the loading forces were floating between positive and negative limit values (see Fig. 2). At the start of fatigue test, each specimen was loaded statically at the maximum and minimum values of the loads that would be applied during the fatigue test. In the case of specimens with reinforcing triangular haunches the absolute values of maximum (positive) and minimum (negative) loading forces were different, i.e. the normal stresses in the observed points of the stringer web (see Fig. 2a) oscillated around the mean stress different from zero. In the case of specimens without haunches the absolute values of maximum (positive) and minimum (negative) loading forces were the same, i.e. the normal stresses in the observed points of the stringer web (see Fig. 2b) oscillated around the mean stress close to zero.

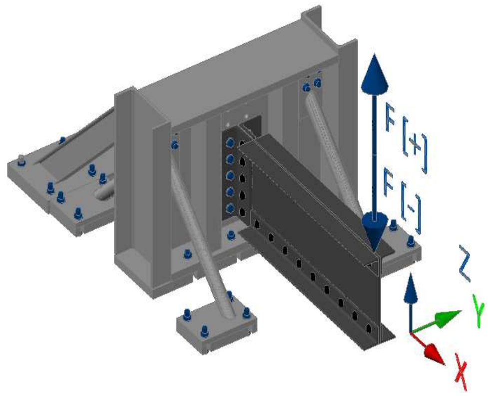

Fig. 3. Configuration of a laboratory specimen and the pulsating forces.

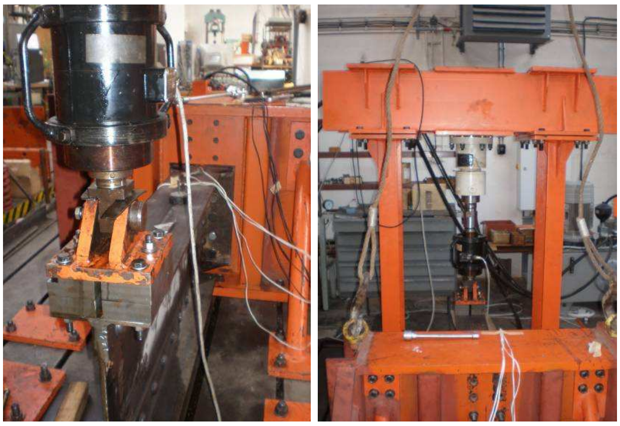

Fig. 4. Illustration photos from fatigue tests.

Normal stresses in the stringer web were measured by means of gauges LY11-6/120 manufactured by Hottinger – Baldwin Messtechnik [3]. Location of gauges is shown in Fig. 2. Fatigue test arrangement is shown in Fig. 3 and in Fig. 4.

3. Experimental Result

3.1. Results of Experimental Fatigue Tests

Typical failures of the investigated detail are presented in Fig. 5. Picture on the left side shows the out-of-roundness of the rivet hole due to material cyclic plasticity after which the beam global deflection is rapidly growing. The picture on the right side shows the common type of fatigue failure in the form of crack starting from the rivet hole.

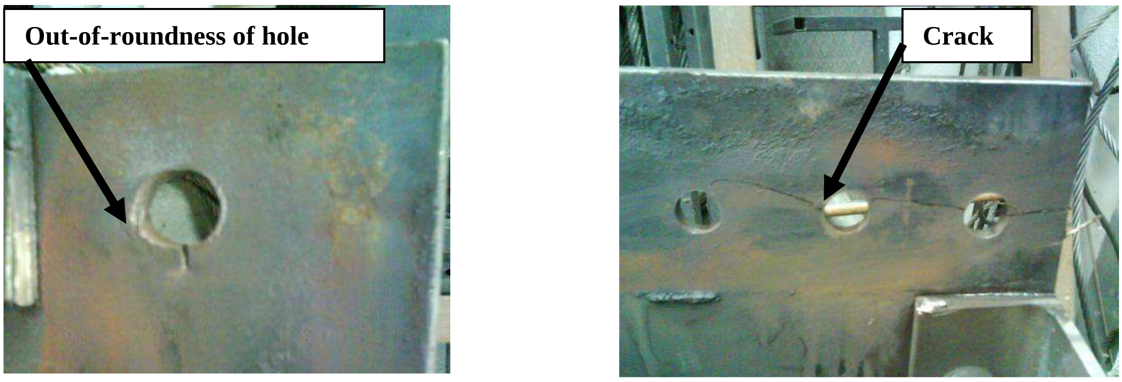

Fig. 5. Types of detail fatigue failures.

The results of the experimental fatigue tests of the specimens described above are presented in the form of stress ranges and corresponding numbers of cycles to failure in Table 1. Specimens No. 1 - 6 had been tested previously, specimens of No. 7 - 10 represent the results of the last tests in 2010. As it was mentioned above, eleven specimens altogether were experimentally investigated, but one specimen failed due to cracks in connecting angle, so that its result had to be rejected. In order to avoid this type of failure, the remaining four specimens were reinforced by means of triangular sheets welded between angle flanges.

Different form of the cracks in the real bridge stringer webs (see Fig. 1) and cracks of the tested specimens (see Fig.5) was caused due to different stringer static system. While stringer in Fig. 1 seems to be quasi simple supported beam, the static system of the tested stringers is cantilever (see Fig. 2 and Fig.3).

Specimen No.	Equivalent stress range $\Delta\sigma_e$ [MPa]	Number of cycles to failure N [cycles]
1	114,9	1 276 750
2	136,6	571 000
3	88,0	3 653 000
4	97,2	2 218 900
5	147,7	629 000
6	125,4	798 350
7	121,7	1 240 450
8	116,3	1 863 760
9	142,5	1 406 080
10	119,0	1 697 600

Tab.1. Results of fatigue tests.

Based on the regressive analysis results, the fatigue resistance of investigated detail corresponding to 2 x 10^6 cycles is $\Delta\sigma_C$ = 83.9 MPa. Therefore, the detail can be classified to the category 80 with characteristic fatigue strength $\Delta\sigma_C$ = 80 MPa in accordance with [1].

4. Conclusion

The paper deals with experimental investigation of the fatigue resistance of the stringer to cross-beam riveted connection representing the common fatigue prone structural detail of existing steel railway bridges with open decks. After collection of all experimental data from tests realized in 2009 and 2010 and based on the obtained results of all performed fatigue tests, the detail could be classified to the category 80 according to EN 1993-1-9 [1].

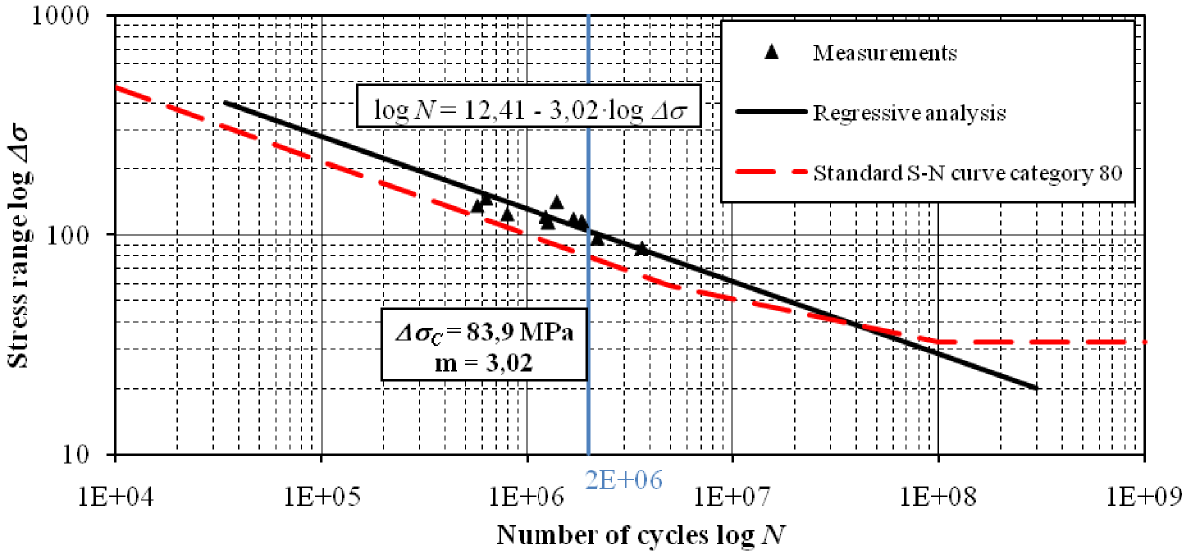

Fig. 6. Results of fatigue tests.

Acknowledgement

This work was supported by the Slovak Research and Development Agency under the contract No. SUSPP-0005-07 and by the Scientific Grant Agency of the Slovak Republic under the project 1/0311/09 "Durability of the steel bridge structural members".

References

[1] EN 1993-1-9, EUROCODE 3, *Design of steel structures, Part 1.9, Fatigue*, CEN, Brussels 2003.

[2] GOCÁL, J., VIČAN, J., HLINKA, R., JOŠT, J., "*Laboratory tests of a typical fatigue prone riveted steel railway bridge structural detail*", Procedia Engineering 2 (2010), ScienceDirect, Elsevier 2010, pp. 1761–1766.

[3] TREBUŇA, R., ŠIMČÁK, F., *Handbook of Experimental Mechanics*, Edition of the scientific and technical literature, Košice, 2007.

[4] VIČAN, J., GOCÁL, J., MELIŠ, B., KOTEŠ, P., KOTULA, P., "*Real Behaviour and Remaining Lifetime of Bridge Structures*", Communications - Scientific Letters of the University of Žilina, Vol. 2/2008, [Žilina: EDIS - ŽU Žilina], ISSN 1335-4205, p. 30-37.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Determining Pearson's Linear Correlation Coefficient in order to Establish the Relation between the Value of the Leachate Water Reaction and the Methane Emission from a Domestic Waste Disposal Site

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Abstract. Waste disposal sites are treated as bioreactors in which occur various processes aiming at a planned and controlled stabilization of the disposed refuse weight. The objective of the research was to define the level and direction of Pearson's linear correlation dependency between the pH value of the leachate water and the level of the methane emission (percentage of volume). Pearson's linear correlation coefficient (*r*), defined by the covariance standardization, is a tool to measure the level of the straight-line reaction between two numerical characteristics. The research was conducted on the basis of the results of 26 trials performed as a part of the monitoring examination of the waste disposal site in Promnik, municipality of Strawczyn in the years 2003 – 2009. In the first stage of recognizing the characteristics of the examined waste disposal site it was assumed that these characteristics are: pH value of the leachate water and the methane (CH_4) emission level. The conducted calculations demonstrated negative correlation dependency between the examined variables. In two percent of cases the changes of one characteristic i.e. the one defining CH4 level are conditioned by changes of the other characteristic i.e. the level of pH value. The result of correlating the examined parameters is unsatisfactory, therefore the next steps should aim at searching different parameters that can serve as an explanatory variable for the methane emission level i.e. the response variable.

Keywords: Pearson's linear correlation dependency, covariance standardization, negative correlation dependency, positive correlation dependency.

1. Introduction

This paper deals with some of the findings of the research conducted on ten waste disposal sites in the Świętokrzyskie province.

A waste disposal site is a structure localized and designed according to building principles and regulations. The decay of the solid waste disposed of in the sites is influenced by chemical, physical and biological processes. The decay generates solid, liquid and gaseous substances, most of which may pose threat to the environment. Chemical reactions taking place on the site include the following processes: hydrolysis, dissolution, precipitation, chemical weathering, sorption, ion exchange, and desorption [1].

Physical-chemical processes play an important role during the waste stabilization. The main factor, however, that determines the degree and intensity of the waste decay, is the microbiological activity of the system dependent on easily decomposed organic matter, as well as on water, oxygen and light. Biological processes on the waste disposal site proceed in stages, each of them requiring proper environment and substrate, and giving final products with their characteristic features. Reference sources present the division of reactions that take place on the waste disposal site in the form of three to five stages [2,3]. The division of stages with regard to the physical-chemical composition of leachates and biogas is done on the basis of the following assumption [4,5,8,]:

- stage I – initial -- a large amount of easily biodegradable organic matter in the waste and the existing oxygen conditions in the waste deposit lead to the high concentration of organic substances in the leachates;
- stage II – acidegenesis -- the leachates still have the high concentration of organic matter in them, volatile fatty acids (VFA) are produced in an intensive manner, the values of pH are low, the anaerobic bacteria manifest no activity, methane is practically not detectable in the gas;
- stage III - methanogenic unstable – the increase of pH, the decrease of redox potential to the negative values, a distinct fall of VFA concentration, the growth of the anaerobic bacteria activity, the rise of the methane concentration in the gas up to the values typical of the methanonogenic stable stage;
- Stage IV – methanogenic stable -- relatively steady concentration of organic matter in the leachates, the growth of the redox potential, the methane content in the gas stays at a relatively steady and high level – about 60-70%.

In many countries waste disposal sites serve as the places to deposit a whole lot of refuse with high percentage of organic matter. Therefore they are treated as a kind of "bioreactor", where the application of particular technologies results in processes aiming at a planned and controlled stabilization of the disposed refuse weight and of the emitted pollution [12,13,15].

Waste disposal sites are operated in accordance with the legal regulations and it justifies treating these objects as gigantic bioreactors. Hence there is a ground for searching correlations between parameters describing the stabilization condition of the leachate from the fermented organic matter contained in the refuse and the percentage of components in the biogas.

2. Materials and Methods

Many measuring instruments can be used to express in figures the degree of interdependence between the two variables. Their selection depends i.a. on the kind of characteristics (measurable, immeasurable, combined) whose interdependence is examined, as well as on the number of observations (correlation table, correlation series) and on the form the dependency takes (straight-line or curved-line regression)[10]. One of the measuring tools for the correlation degree is Pearson's linear correlation coefficient [7,9,10].

This coefficient (r_{xy}) is a tool to measure the degree of the straight-line relation between the two numerical characteristics. In the first stage of recognizing the characteristics of the waste disposal site under examination, a hypothesis is assumed that those characteristics are: the pH value of the leachate water and the level of the methane (CH_4) emission.

The formula for Pearson's linear correlation coefficient is defined by the covariance standardization. The covariance is the arithmetic mean of the product of the deviations of the values of the variables X and Y from their arithmetic mean.

for individual data (in the form of the correlation series)

$$\operatorname{cov}(x,y)=\operatorname{cov}(y,x)=\frac{1}{n}\sum_{i=1}^{n}(x_i-\bar{x})(y_i-\bar{y})=\overline{xy}-\bar{x}\cdot\bar{y} \qquad (1)$$

for the data listed in the form of the correlation table

$$\operatorname{cov}(x,y)=\operatorname{cov}(y,x)=\frac{1}{n}\sum_{i=1}^{k}\sum_{j=1}^{r}(x_i-\bar{x})(y_i-\bar{y})n_{ij}=\overline{xy}-\bar{x}\cdot\bar{y} \qquad (2)$$

for the individual data

$$\overline{xy}=\frac{1}{n}\sum_{i=1}^{n}x_i y_i \qquad (3)$$

for the correlation table

$$\overline{xy}=\frac{1}{n}\sum_{i=1}^{k}\sum_{j=1}^{r}x_i y_j n_{ij} \qquad (4)$$

The covariance communicates the following information about the correlation dependency:

cov(x,y) = 0 – lack of correlation dependency;

cov(x,y) < 0 – negative correlation dependency,

cov(x,y) > 0 – positive correlation dependency,

The covariance characterizes the interchangeability of the examined data, but its value depends on the order of magnitude in which both characteristics are expressed. This means that it cannot be used directly to the comparison.

Pearson's linear correlation coefficient, defined by the covariance standardization, is a standardized tool to measure the intensity and direction of the linear interdependence of the two measurable variables X and Y:

$$r_{xy} = r_{yx} = \frac{\mathrm{cov}(x, y)}{s(x)s(y)} \qquad (5)$$

Pearson's linear correlation coefficient is a standardized measurement, assuming the values from the bracket: $-1 \leq r_{xy} \leq +1$.

The positive value of the coefficient indicates the existence of the positive interdependence, whereas the negative value indicates the existence of the negative interdependence. The closer the absolute value of the coefficient approaches one, the stronger the correlation dependency between the examined variables is [10].

The correlation coefficient square is called the determination coefficient provides the information to what extent the dependent variable (effect) is explained by the explanatory variable (cause).

3. Results

The degree and direction of Pearson's linear correlation dependency between the pH value of the leachate waters and the level of the methane emission (percentage of the volume) was defined on the basis of the results of 26 trials performed as a part of the monitoring examination [6] of the waste disposal site in Promnik, municipality of Strawczyn in the years 2003 – 2009. This disposal facility provides services for the agglomeration of Kielce.

On the basis of the point (correlation) diagram analysis (Fig.1) it can be stated that the dependency between the examined variables has a straight-line character. Therefore the degree and direction of the dependency can be evaluated with the use of Pearson's linear correlation coefficient.

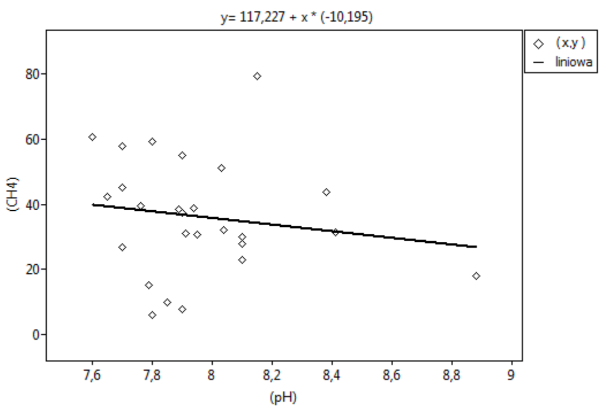

Fig. 1. Point correlation diagram.

x_i (pH)	y_i (CH_4)	$x_i - \overline{x}$	$y_i - \overline{y}$	$(x_i - \overline{x})(y_i - \overline{y})$	$(x_i - \overline{x})^2$	$(y_i - \overline{y})^2$
7,80	59,07	-0,15	22,98	-3,45	0,02	528,08
8,10	30,01	0,15	-6,08	-0,91	0,02	36,97
7,70	26,76	-0,25	-9,33	2,33	0,05	87,05
8,10	27,87	0,15	-8,22	-1,23	0,02	67,57
8,10	23,01	0,15	-13,08	-1,96	0,02	171,09
7,80	6,03	-0,15	-30,06	4,51	0,02	903,60
7,60	60,71	-0,35	24,62	-8,62	0,12	606,14
7,90	37,26	-0,05	1,17	-0,06	0,0025	1,37
7,90	54,91	-0,05	18,82	-0,94	0,0025	354,19
7,70	57,93	-0,25	21,84	-5,46	0,06	476,99
7,94	38,87	-0,01	2,78	-0,03	0,0001	7,73
7,89	38,43	-0,06	2,34	-0,14	0,0036	5,48
8,88	18,10	0,93	-17,99	-16,73	0,86	323,64
7,90	7,95	-0,05	-28,14	1,41	0,0025	791,86
7,85	10,00	-0,10	-26,09	2,61	0,01	680,69
8,38	43,60	0,43	7,51	3,23	0,18	56,40
8,41	31,25	0,46	-4,84	-2,23	0,21	23,43
7,95	30,85	0	-5,24	0	0	27,46
7,79	15,27	-0,16	-20,82	3,33	0,02	433,47
7,70	45,10	-0,25	9,01	-2,25	0,06	81,18
7,91	30,87	-0,04	-5,22	0,21	0,0016	27,25
8,03	51,10	0,08	15,01	1,20	0,0064	225,30
7,76	39,55	-0,19	3,46	-0,66	0,04	11,97
8,04	31,95	0,09	-4,14	-0,37	0,01	17,14
8,15	79,30	0,20	43,21	8,64	0,04	1867,10
7,65	42,47	-0,30	6,38	-1,91	0,09	40,70
206,93	**938,22**	**x**	**x**	**-19,48**	**1,8692**	**7853,85**

Tab. 1. Correlation table with the results of the monitoring research conducted in Promnik (in accordance with the "Environmental Ministry Regulation on the scope, time, manner, and conditions for monitoring landfills (Journal of Laws, No 220 item 1858 as amended).

The mean pH value for the leachate waters is $\overline{x} = 206{,}93 : 26 = 7{,}95$

The mean methane emission is $\overline{y} = 938{,}22 : 26 = 36{,}09$

In order to calculate Pearson's linear correlation coefficient it is necessary to know the standard deviations of the two characteristics:

The standard deviation of the pH value equals:

$$s(x) = \sqrt{\frac{\sum_{i=1}^{n}(x_i - \overline{x})^2}{n}} = \sqrt{\frac{1{,}8692}{26}} = 0{,}26 \qquad (6)$$

The standard deviation of the methane emission equals:

$$s(y) = \sqrt{\frac{\sum_{i=1}^{n}(y_i - \overline{y})^2}{n}} = \sqrt{\frac{7853{,}85}{26}} = 17{,}38 \qquad (7)$$

With the above information it is possibile to calculate Pearson's linear correlation coefficient:

$$r_{xy} = \frac{-19{,}48}{26 \cdot 0{,}26 \cdot 17{,}38} = -0{,}1658 \qquad (8)$$

$$r_{xy}^2 = 0{,}0275 \qquad (9)$$

The obtained result indicates that between the examined variables there is a negative correlation dependency. The negative dependency means that the increase of one variable results is accompanied by the decrease of other variable results, which could have been expected.

In practice, on the waste disposal site, the refuse is decomposed in a nonaerobic process [11,14]. Various populations of bacteria hydrolize polysaccharides, proteins and fats into simpler compounds (volatile fatty acids, alcohols, sugars, aminoacids). The resultant compounds are decomposed into organic acids, e.g. formic, acetic, propanoic acid, butyric, valeric, hexanoic acids, alcohols, e.g. metanol, ethanol, aldehyde, gaseous products .
The high concentration of the dissolved carbon dioxide and organic acids in the leachate contributes to a significant pH decrease of the leachate to the value 5.5 to 6.5, which in turn makes other organic and inorganic substances contained in the refuse transit into soluble forms [11]. At this stage the leachate waters are strongly chemically aggressive, the methane production is limited. The solubility of organic and inorganic compounds, as well as the effective functioning of anaerobic bacteria depend on the activity of hydronium ions [H_3O^+]. The proper pH value for the substrate decomposing bacteria falls within the range of 5.2 to 6.3, whereas methane bacteria require constant neutral environment: pH 6.8 to 7.2.
Slight changes in pH cause the inhibition of the methane bacteria multiplication and the fall of their population, which results in the low methane emission [5].

4. Conclusion

The conducted calculations of the correlation coefficient r_{xy}^2 according to the dependency (9) show that in two percent of cases changes of one characteristic defining CH4 level are conditioned by changes of the other characteristic i.e. the level of pH value. The result of correlating the examined parameters is unsatisfactory, therefore the next steps should aim at searching different parameters that can serve as an explanatory variable for the methane emission level i.e. the response variable.

According to the reference sources the following factors also have a significant influence on the methane production: temperature, methane bacteria inhibitors, the redox potential and others. As far as the data from the monitoring research is concerned, further parameters will be examined according to a rolling plan. Those parameters, acting as a defining variable, will significantly contribute to enhancing the degree of correlation.

References

[1] McBean E.A, Rovers F.A., Farquhar G.J., Solid waste landfill engineering and design, Prentice Hall PTR, New Jersey 1995.

[2] Jędrczak A., Biologiczne przetwarzanie odpadów, Wydawnictwo Naukowe PWN 2007.

[3] Nicolas J, Themelis, P. Ulloa A.,: Methane generation in landfills, Renewable Energy 32 (2007), 1243 – 1257.

[4] Kaczorek K., Zieleniewska A., Krzystek L., Ledakowicz S., Wpływ pogłębionego utleniania i recyrkulacji na biodegradację odcieków ze składowisk odpadów, Inżynieria Chemiczna i Procesowa 2004: 1041-1046.

[5] Chugh S., Clarke W., Pullammanappallil P., Rudolph V., Effect of recalculated leachate volume on MSW degradation, Waste Menagement 1998: 564-73.

[6] Rozporządzenie Ministra Środowiska z dnia 9 grudnia 2002 r. w sprawie zakresu, czasu, sposobu oraz warunków prowadzenia monitoringu składowisk odpadów (Dz. U. Nr 220. poz. 1858 ze zm).

[7] Huan-jung Fan, Hung-Yee Shu, Hsin-Sin Yang, Wen-Ching Chen, Characteristics of landfill leachates in central Taiwan, Science of the Total Environment 361 (2006) 25-37.

[8] Wei Li, Tao Hua, Qixing Zhou, Shuguang Zhang, Fengxiang Li, Treatment of stabilized landfill leachate by the combined process of coagulation/flocculation and powder activated carbon adsorption, Desalination 264 (2010) 56-62

[9] Tatsi A.A, Zouboulis A.I., A field investigation of the quantity and quality of leachate from a municipal solid waste landfill in a Mediterranean climate (Thessaloniki, Greece), Advences in Environmental Research 6 (2002) 207 -219

[10] Fisz M., Rachunek prawdopodobieństwa i statystyka matematyczna, PWN, Warszawa, 1969, s.98.

[11] Chan G.Y.S,. Chu L.M, Wong M.H.., Effects of leachate recirculation on biogas production from landfill co-disposal of municipal solid waste, sewage sludge and marine, Environmental Pollution 118 (2002) 393-399

[12] Houhu Zhang, Pinjing He, Liming Shao., Methane emission from MSW landfill with sandy soil covers under leachate recirculation and subsurface irigation, Atmospheric Environmental 42 (2008) 5579-5588

[13] Mostafa Waritch., Bioreactor landfills: experimental and field results, Waste Managament 22 (2002) 7-17

[14] Mutasem El-Fadel, Angelos N. Findikakis and James O. Leckie., Environmental impacts of solid waste landfiling, Journal of Environmental Managament 50 (1997) 1-25

[15] Xian Qu, Pin-Jing He, Li-Ming Shao, Duu-Jong Lee., Heavy metals mobility in full-scale bioreactor landfill: Initial stage, Chemosphere 70 (2008) 769 – 777

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Thermal Effects on Airfield Surfaces

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Abstract. Airfield surfaces, as a special type of concrete structures, are prone to negative effects of increased temperatures of exhaust gases discharged from the nozzles of air vehicles taking-off. The complex chemical composition as well as the high temperatures of the products of incineration coming from airplanes' nozzles affect the deterioration of the conditions of operation of the surface and decrease its durability. Surfaces and materials used for preparation of concrete mixes intended for the flat surfaces of airstrips were subjected to laboratory tests. Based on the results, a concrete mixture with modified contents, characterized by increased resistance to high temperatures has been designed.

Keywords: Airfield surfaces, temperature, concrete.

1. Thermal Effects on Airfield Surfaces

Thermal processes caused by internal heat sources in concrete as well as changes of atmospheric conditions, and in particular deviations of temperature, both daily and annual, have a negative impact on airfield surfaces. Thus, attributing to occurrence of very advanced damage.

An additional factor, which is very detrimental to surfaces, are the enforced thermal loads caused by hot products of incineration coming from engine nozzles of airplanes. Airstreams discharged from airplane nozzles are characterized by diversified, many times repeated, short-term, pulse-like and local effects on the surface. The area of especially destructive effects of the stream of hot discharge gases on the surface of the airfield is the so-called "stream core", with characteristics determined by the properties of the plane's propelling system. [1] As the greatest temperatures of the stream of hot gases may be observed at the extension of the engine's axis, and the further from it the temperatures decrease, the inclination of the longitudinal axis of the plane in relation to the surface level is very important. Increasing the inclination from 0° to 16° results in intensification of the temperature affecting the surface from ca. 140°C to over 520°C. [2]

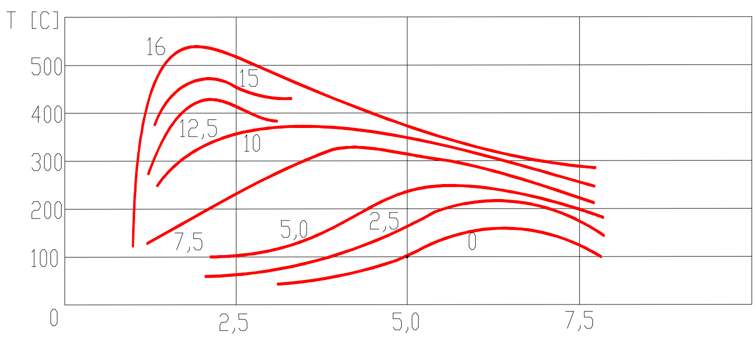

Fig. 1. Chart of temperatures in the upper layer of the surface made of cement concrete with a constant height of nozzles above the surface (38 cm) depending on the inclination of the engine's nozzle in relation to the surface level [2].

The effect of increased temperature is distributed in the range of 70 to 90 m behind the edge of the plane's nozzle, resulting in local heating of the airstrip exceeding 200°C. Deformation of the stream of gases, caused by blows of side wind is an often effect, which increases along with the distance of the stream from the engine's nozzle [1].

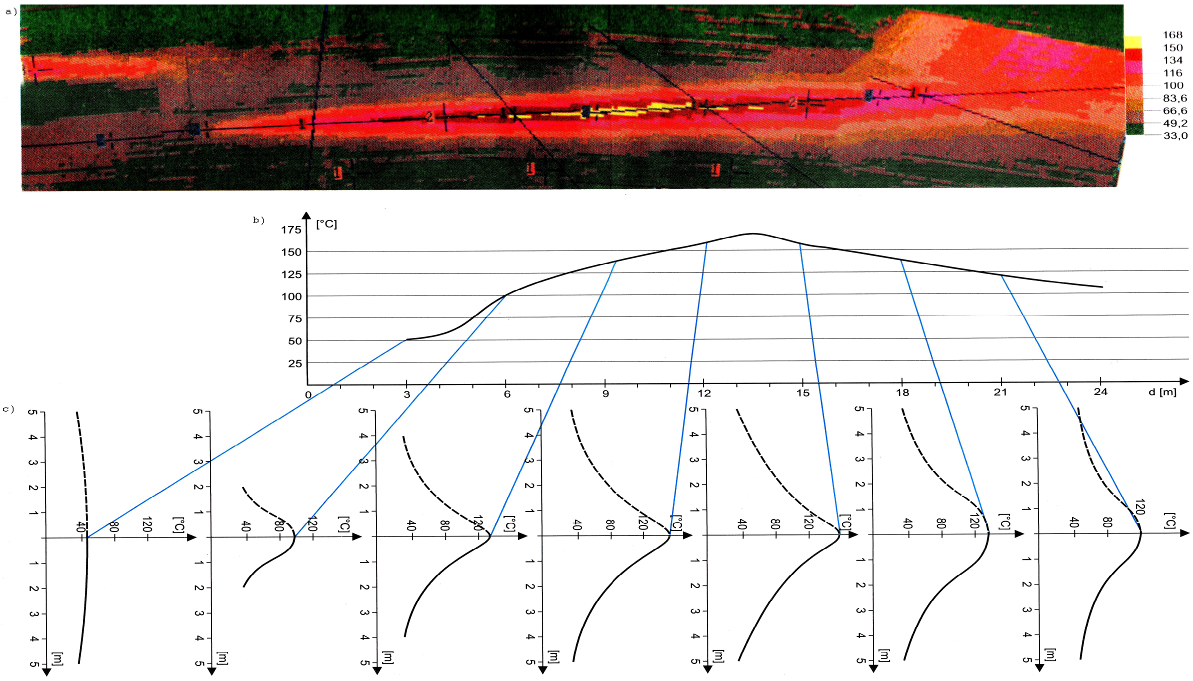

Fig. 2. Thermal state of the surface behind a jet's nozzle, a – a thermogram of the surface after 12 minutes of heating, b-linear longitudinal section through the stream of gases, c – stream of gases in cross-sections to the axis of the stream of gases [1].

The surface slab, which must transfer static and dynamic loads from the moving planes as well as thermal loads is exposed to freezing and de-freezing of the surface layer of the structure. Destructive effects of water and frost, especially in the sections of the surface, which are most exposed to the effects of thermal enforced loads, attributes to surface flaking in the near-surface layer [2]. This damage is observed especially in the region of rainwater reservoir, in the region of significant traffic intensity, and in particular in the band of runways' axis The starting form of occurrence of such type of damage is the presence of plates 1-3 mm. With time and if left without repairs, the plates increase in size in extensive areas (even a dozen m^2), and while disconnecting the concrete material layer by layer they go deeper into the slab to greater depths. [3] Such damage is the most difficult and most dangerous process of surface degradation. Repairs require conscientiousness of performance and high properties of the adopted materials. In this aspect, the necessity of modifying the content of concrete intended for airfield surfaces becomes necessary, and in particular in the case of fragments, which are most exposed to the effects of high temperature of exhaust gases.

2. Laboratory Tests

The purpose of the first stage of laboratory tests was the selection of comparative concrete's components. Typical surface concrete composition, intended for machine placement, adopted in airfields, was used to prepare concrete samples. [4] Thick aggregate, in each leaven, was: three-fraction granite grit: 16-32 mm, 8-16 mm, 2-8 mm, fine aggregate was flushed sand. The selection of cement, aerating agent and plasticizer was made based on tests of 6 sample leavens with the use of various components, the contents of which are presented in table no. 1

Concrete		B 1	B 2	B 3	B 4	B 5	B 6
Granite grit fraction 16/22 mm		+	+	+	+	+	+
Granite grit fraction 8/16 mm		+	+	+	+	+	+
Granite grit fraction 2/8 mm		+	+	+	+	+	+
Flushed sand fraction 0/2 mm		+	+	+	+	+	+
Cement	C 1	+	+				
	C 2			+	+		
	C 3					+	+
Plasticizer	P 1	+		+		+	
	P 2		+		+		+
Aerating agent	Śn 1	+		+		+	
	Śn 2		+		+		+

Tab. 1. Material composition of concrete mixes, concrete class C40/50.

After preparation of leavens, the content of air and consistency were checked, and then, pursuant to the recommendations of the standard, samples were maintained, after 7 and 28 days of maturing durability, resistance to freezing and absorptivity were checked.

Concrete	B 1	B 2	B 3	B 4	B 5	B 6
Average air content from 12 samples [%]	4,2	4,35	5,9	5,02	4,8	5,2
Average crushing resistance after 7 days from 6 samples [MPa]	38,9	39,4	38	38,6	41,6	41,8
Average crushing resistance after 28 days from 6 samples [MPa]	54,9	54,8	54,7	54,95	53,26	55,3
c/w coefficient	0,39	0,39	0,39	0,39	0,39	0,39

Tab. 2. Average results of air content, crushing resistance after 7 and 28 days, freezing resistance and absorptivity for concrete class C40/50.

The results from own laboratory tests were used as base for selection of a more optimal composition of the comparative concrete mixtures (concrete no. 6), which is to be intended for airfield surfaces, and in particular the section of it, which are directly exposed to the destructuve effects from the streams of hot exhaust gases from airplane nozzles. Further tests (second stage) were intended to design a modified composition of the concrete mixture intended for airfield surface, the structure of which is to meet the requirements of operation in increased thermal conditions. Modification of the composition was based on replacing a part of volume of fine aggregate with grog fraction 0-1 and 1-2mm.

Grog is a ceramic material produced by sintering and grinding cauterized fire-resistant clay. Products with an addition of grit are characterized by high resistance to quick temperature changes, meaning effects close to those occurring on airfield surfaces.

Three mixtures differing in terms of content of grog were designed and prepared. The first M7 mixture had 7%, the second B11 – 11%, and the third B15 – 15% of grog compared to the sand content. The results of tests are shown in the table below.

Concrete	B 7	B 11	B 15
Average air content from 12 samples [%]	4,6	4,4	4,8
Consistency rate Ve-Be [s]	14	14	15
w/c coefficient	0,382	0,368	0,355
c/w coefficient	2,621	2,714	2,815
Average crushing resistance after 7 days from 6 samples [MPa]	55,58	59,75	56,58
Average crushing resistance after 28 days from 6 samples [MPa]	63,97	61,48	67,5
Average crushing resistance after 76 days from 6 samples [MPa]	67,65	68,98	71,20
Average pulling force while cracking from 6 samples [kN]	163,6	144,85	150,57

Tab. 3. Average test results for surface concrete with an addition of grog class C40/50.

The tests carried out in the third stage covered determination of the optimal content of grog affecting the increase of resistance of surface concrete exposed to high temperatures emitted from the nozzles of taking-off planes. The optimal addition of grog, determined based on statistical analysis, amounts to 10,8% compared to sand volume. Current tests, the fourth stage, are to analyze the effects in the macro-structure and micro-structures of solidified concrete with an addition of grog subjected to high temperatures.

3. Conclusions

Based on surface concrete tests with and without addition of grog, one may formulate the following conclusions:

- By replacing sand with grog the mixture's water requirements decrease, but one needs to add more aerating additives,
- Resistance to crushing of the concrete with an addition of grog is higher than the resistance to crushing of standard surface concrete,
- Application of a concrete mixture with an addition of grog results in a growth of resistance to crushing the longer the samples maturate,
- The addition of grog in concrete mixtures results in increasing the resistance of concrete to high temperatures,
- Modified composition of surface concrete will allow extending the surface's operation period.

References

[1] NITA P., 2008. *Budowa i utrzymanie nawierzchni lotniskowych*, Warsaw 2008

[2] *„Problemy badań i eksploatacji techniki lotnicze" j. Volume 5,* Warsaw 2006

[3] Nita P. „*Badanie skutków oddziaływań termicznych samolotów na nawierzchnie betonowe*", Drogownictwo, issue 4, 1993, p. from 77 – to 80

[4] Wojtawicki J. „*Przyczyny powstania uszkodzeń nawierzchni z betonu cementowego*", Lotnisko, issue 3, 2008.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Pavement Management System in Slovak Republic and Abroad

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Abstract. The safe and economical infrastructure should be one of the basic task of each country. The effective reconstruction and maintainance of roadways is the basis. This article deals with actual condition of the system of roadway management considering on reconstructionss and maintainance of roadways and with the program equipment in Slovak republik, Czech Republik and in England and their mutual comparisions.

Keywords: HDM-4, ISEH, C920, RoSy PMS, Highways Agency Pavement Management System, SWEEP.N, SWEEP. S.

1. Introduction

A lot of financial resources is used for building, reconstructions and road maintenance and it is important to use them efficiently.[4] For effective use of these resources and right decisions about road maintenance and reconstruction it is needed to have disquisitional information about the condition of road system and about the rare of its disturbances. And also the data-bank is needed, which manages and works up the information. An important part of this is systems and program ambient, which analyses this information and form the necessity and the order of insistence of reconstruction and road maintenance.

2. Pavement Management System in Slovak Republic

This system uses the program: Integrated system of roadway management, part: the basic program. As other similar systems this system uses good deal of input information for functions. The road data-bank provides crucial one.

For operation of this system, mainly these activities are important:

- data acquisition about the roadways surface of chosen sections by exact visual inspections and their analyse,
- measuration of lengtwise and crosswise irregularities of roadways surface by diagnostic apparatus PROFILOGRAPH GE,
- measuration of roadway deflection by dynamic stabilizing testing by diagnostic apparatus FWD KUAB for designation of roadway carrying capacity,
- accomplishment of other required data about chosen sections into designating carts of inputs,
- analyse of roadway surface in term of carrying capacity,
- proposal of a technology and a price of structural retreatment with completing into designating carts,
- analyse of economical effectivity,
- arrangement of building order for roadway reparations[1]

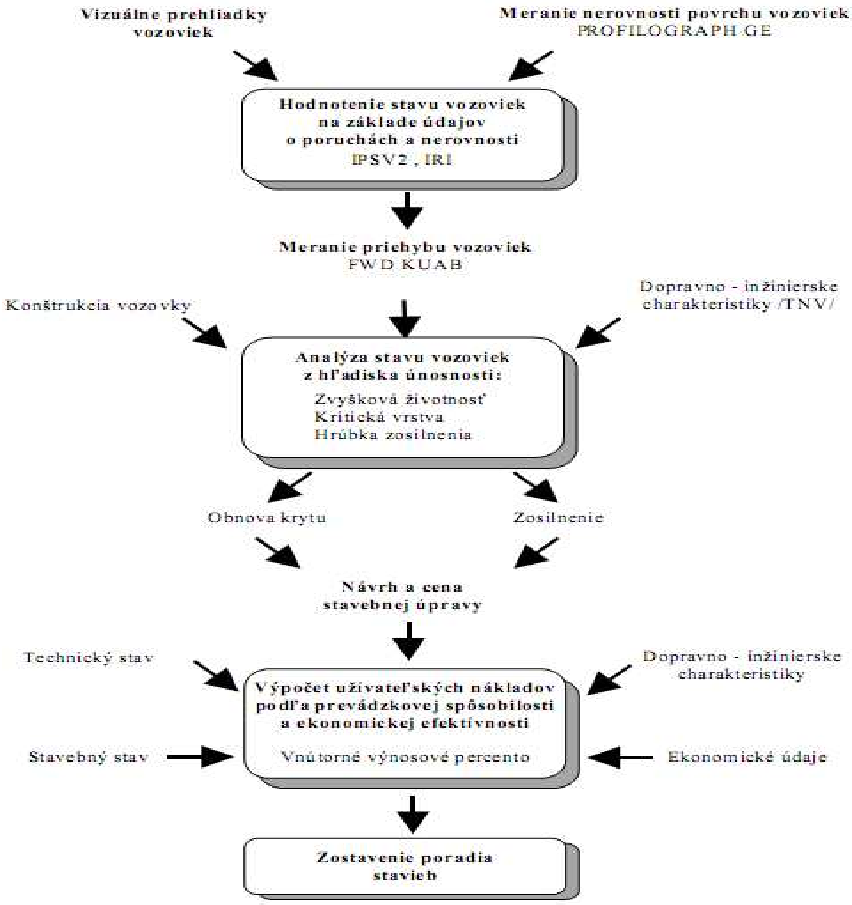

Fig. 1. Decision-making process in proposal of order for rehabilitation[2,3].

Momentary there is an autonomous sofoware program in Slovak Republic for:

- evaluation of new building effectivity (C920)
- evaluation of reconstructions and maintenance effetivity (ISEH)
- complex control of the system of roadway management HDM-4

3. Pavement Management System in Czech Republik

Nowadays there is used the program set-up RoSy PMS in Czech Republik.

Fig. 2. An yearly circle SHV RoSy PMS[6].

The system RoSy PMS is enlarged database system and it includes modules for:

- asquisition and elaboration of data
- report of database of variable and non-variable roadway parameters
- calculation of maintenance and reconstuctions plans and their technical-economical optimalization[6]

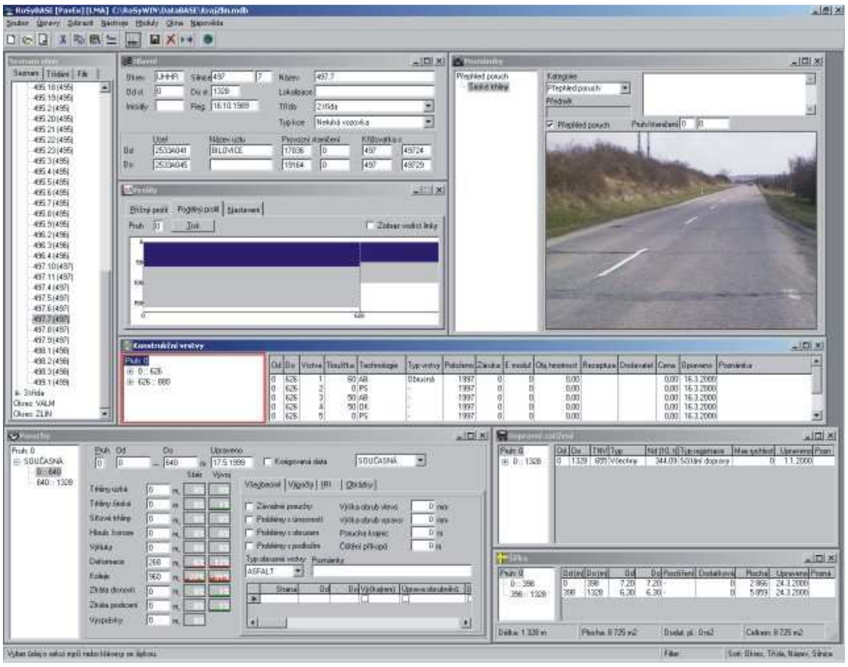

Fig. 3. A sample of the system RoSy PMS[6].

Another device of the system is the modul MAPwhich enablesto form and figure data saved in database of the systém in digital map. These data are saved in standard format ESRI for geogpraphical informative systems and so it is able to use them also apart from this system, for example intra regional or town informative system GIS.

For arrangement of meintenance and reconstructions plans, the system contains another devices:

- degradative models of each watched variable parameter
- registers of technologies of roadway maintenance and reconstructions[6]

Calculation of the plan is practised in agreement with TP 87 Designing of roadway maintenance and reconstructions. This plan is generally medium-term (ten years) and is worked up in two levels:

- Financial plan – optimal measure in optimal time without limitation of financial resources

Fig. 4. A sample of graph of financial plan[6].

- The budget – optimalized measure intra available financial resources with evaluation of impacts on the road system

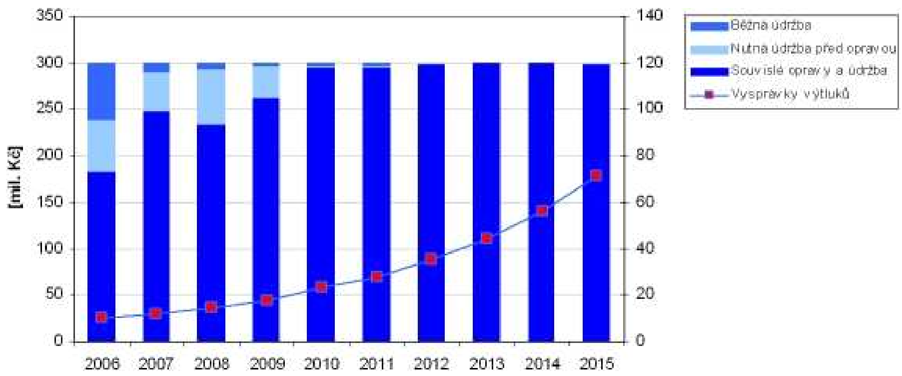

Fig. 5. A sample of graph of the budget[6].

4. Pavement Management System in England

The Highways Agency of the Department for Transport responsible for English highways (7754 kilometers) which takes 25% of conveying charge and 50% of heavy cartage.

HAPMS – HIGHWAYS AGENCY PAVEMENT MANAGEMENT SYSTEM

Historic, the management of informative system intra Highway Agency has been developed indipendently for all fields of exploitation.HAPMS-Highways Agency Pavement Management System started to develop in1998 and its development should costs England 3,5 mil pounds a year [3].

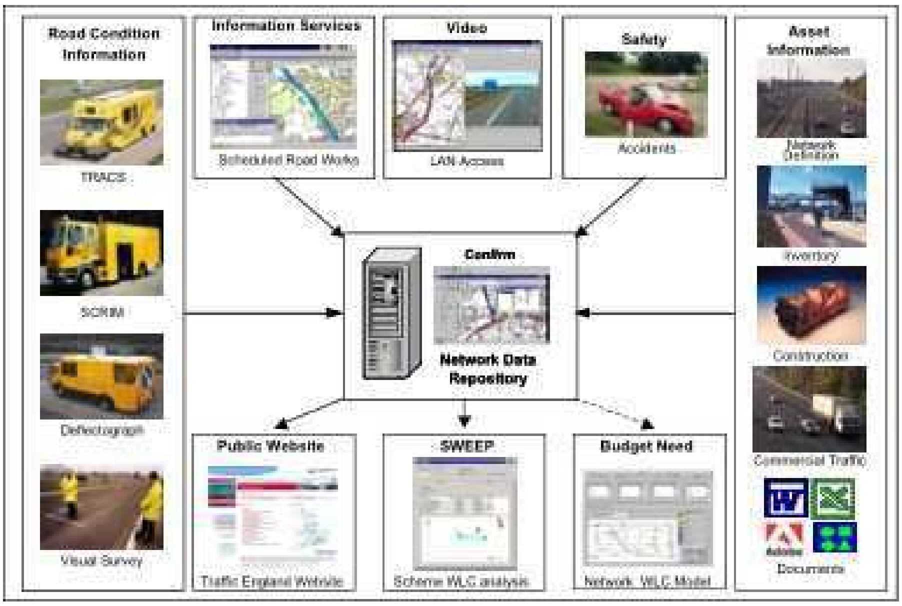

Fig. 6. Highways Agency Pavement Management System (HAMPS) [5].

The HAPMS project was practised so that two programes should be its oputput.SWEEP.N as a network level and SWEEP.S as a project level. SWEEP.S is designed to give solution in roadway maintenance. It is a device which provides analyse of type „what if“, prescribes practiced appreciations for needs of roadway management and so on. It is a program for lifelong economical appreciations in the area of roadways (schemata) and it provides detailed economical analyses of particular schemata abreast of project level. SWEEP.S affords to the user geographical presentation of maintenance, reconstructions and conditions on the road (variable parameters) intra schemata. This program works with the road which is devided into particular cells. The width of a cell is a half of a traffic lane and its length is dependent on the change of conditions on the roadways, intensity of vehicles,geometry of traffic lanes and on facilities of a section (variable parameter). However the highest length of the cell is 10m. These cells form a grate on the road. [5]

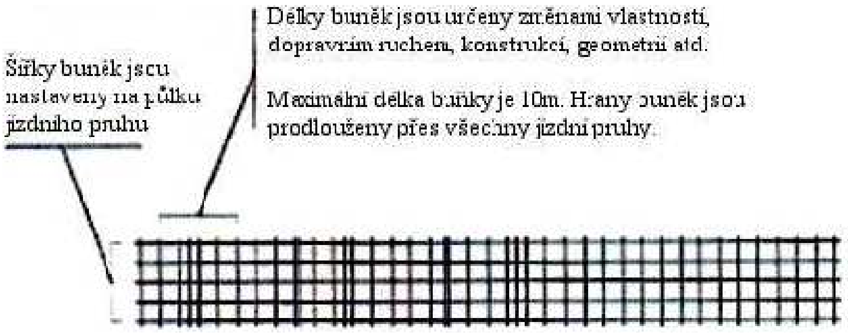

Fig. 7. The schema of roadway dividing on particular cells[5].

Procedure

The program works with scheme of maintenance. These ones have to be deposed and defined individually. SWEEP.S maintains connexion with actual database which ensures that scheme proposed by SEEP.S contains actual data about geometry, structural composition and about variable parameters.

Calculation of the Budget

The budget is calculated by SESM (Scheme elementary segment matrix – the matrix of schemata).The final cost/daration for maintenace or reconstruction is calculated from combination of particular rates.

The basic cost and output rates gained by SWEEP.S vary on the basis of these attributes:
-the type of the road (highway,...)
-the direction of traffic stream (singledirectional, bidirectional)
-the enviroment (rural area, ...)
-the length of labour (at night, 24 hours, during the day,weekend,...)
-the type of roadway construction (asphaltic, concrete,...)
.the type of booking (one lane, both lane,...)

Closing Analyse

Closing analyses are provided by two methods:
-pre-processing
-whole life cost.[5]

5. Conclusion

The system of roadway management in each country is justified differently for conditions of particular countries and also program accessories for evaluation of effectivity of maintenance and reconstructions projects is different. The future of roadways is reliant on accuracy of system settings. These systems should serve as means for correct and effective decision. Each country has its reliable system. The set-up of priorities and basic conditions change together with a change of locality location, climaticconditions, roadcharacteristics,… So it is a job of specialized workers which initial conditions to select as ideal, which parameters to set up as primary. Financial resources selected for road sector have specified upper limit. This situation forces to more rationalexploitation of finances namely for building of new roads, for maintenance and consecutive for repair. As it was mentioned, there is still no system in Czech Republic which would solves the roadway management and so the system of roadway management, as Highway Agency, can be for us a recipe which way to go.

Acknowledgement

This contribution is the result of the project implementation: "Independent Research of Civil Engineering Construction for Increase in Construction Elements Effectiveness" (ITMS: 26220220112) supported by the Research & Development Operational Programme funded by the ERDF.

References

[1] VALUCH, M.: *Metodika systému ekonomického hodnotenia prevádzkovej spôsobilosti a výkonnosti cestných vozoviek* (program SEH PS), ŽU v Žiline, SvF Žilina 1999.

[2] MIKOLAJ, J., a kolektív: *Systém hospodárenia s vozovkou*, VŠDS, 1996.

[3] TP 10/2006, *Systém hospodárenia s vozovkami.*

[4] DECKÝ, M., GAVULOVÁ, A., PUTIRKA, D., PITOŇÁK, M., VANGEL, J., ZGÚTOVÁ, K.: *Navrhovanie a rozpočtovanie asfaltových vozoviek.* Vysokoškolské skriptá, Stavebná fakulta ŽU, 2010, s. 300, ISBN 978-80-970388-0-9.

[5] RADIMSKY, M., HOLDOVA, J.: *Pavement managment system in England.*

[6] http://www.pavex.cz/.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

The Influence of Operational Temperature on Changing of Resilient Modulus of Elasticity of the Asphalt Concrete with Low-Viscosity Modifier

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Abstract. Based on the experiments it was found that the operating temperature significantly influenced on resilient modulus of elasticity the of asphalt concrete. The study was intended for asphalt concrete purposed to wearing course AC 11 S. In the experiment was used a bitumen 35/50 which was modified by means of synthetic wax (Fisher-Tropsh) in a range of 1.5% to 4.0% with steps 0.5%. The asphalt concrete has been compacted at the optimum characteristic temperature of 125 ºC. The study revealed a low susceptibility of temperature of asphalt concrete with asphalt low-viscosity in the range of 5 ºC to 25 ºC. In addition, it was revealed a negative impact of lack compaction level on resilient modulus of elasticity of the asphalt concrete.

Keywords: Synthetic wax, resilient modulus of elasticity, request surface.

1. Introduction

The increase in requirements in relation to the load of road surfaces is a result of increasing in the level of maximum wheel load that are transmitted to the ground and destruction of roads exaggerated by a climatic factor [1]. An important element in lowering the bearing capacity of pavement is decreasing a compaction temperature in the autumn season. Possible solution of this problem is using of mineral mix asphalt produced in the “warm technology” (WMA). The implementation of asphalt concrete in such technologies requires modification of the viscosity of the binder by foaming or chemical modification. Chemical modification is relisted by using synthetic waxes which can substantially reduce the level of viscosity of asphalt at temperatures above 110 º C. An additional advantage of this type of technology is improvement characteristics of visco-elastic asphalt concrete bears on a resilient modulus of elasticity expressed by a thermal susceptibility of the material. Analysis of a resilient modulus of elasticity according to EN 12697-26 [2], in particular temperature ranges, gives a chance of quick evaluation of excessive stiffness of the asphalt concrete revealing in reductions in fatigue life. The study of a resilient modulus of elasticity complements knowledge of the potential rutting of the asphalt concrete.

2. Dynamic Viscosity of Low-Viscosity Bitumen

The tests of the impact of the low – viscosity modifier on changes of the bitumen properties were conducted for the traditional 35/50 bitumen. This binder was modified by low-viscosity modifier in the range of 1,5% to 4,0% with steps 0,5% in relation to the amount of the bitumen. The samples were blended according to the condition of production [3]. Dynamic viscosity expresses the internal friction which occurs due to the existence of cohesion forces between the constituents of bitumen when they moving relative to each other [4]. The viscosity of bitumen was measured at a shear rate of 1 s^{-1}. The viscosity measurement was carried out in three specific temperatures 60 ºC, 90 ºC and 135 ºC.

The changes of dynamic viscosity as a function of temperature was presented in figure 1.

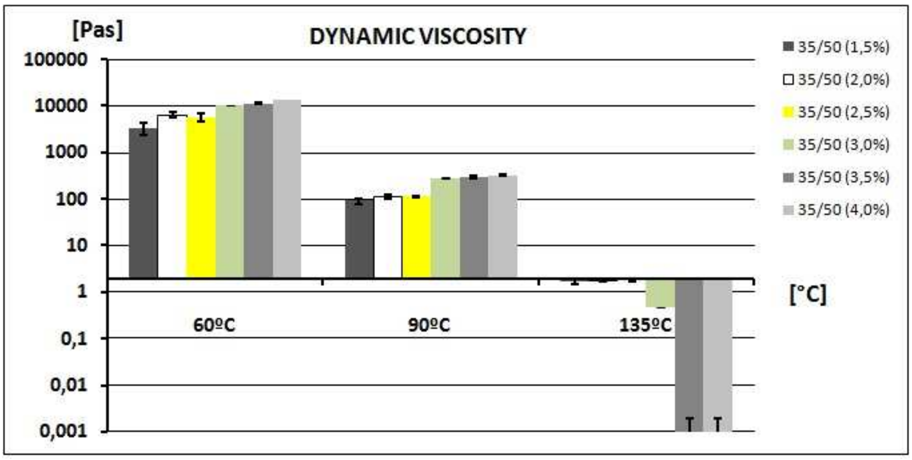


Fig. 1. Dynamic viscosity versus temperature.

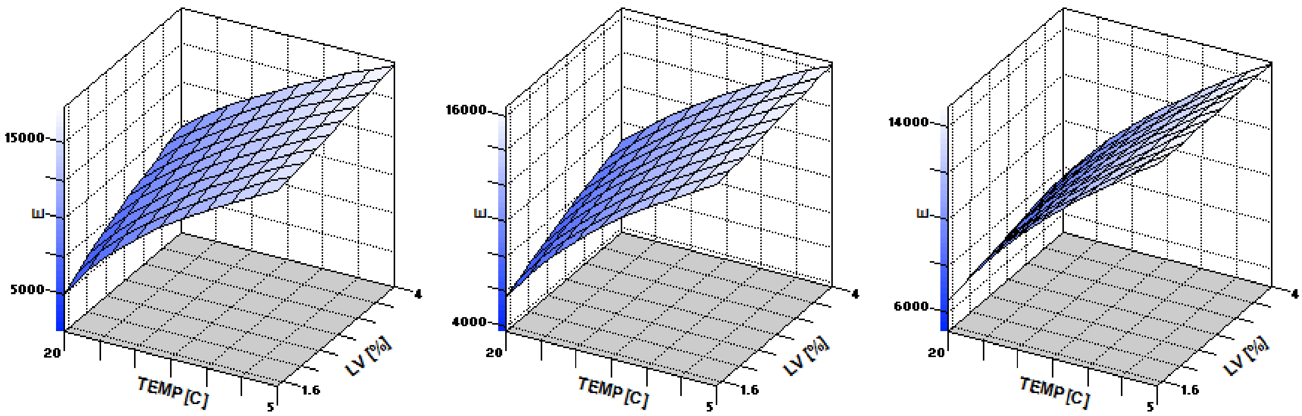
The results show that as content of the low-viscosity modifier increase, there is a considerable diversity in the measurement of dynamic viscosity of bitumen at medium temperatures. It should be noted that as the temperature increases the dynamic viscosity was decreased of the bitumen. With the increasing of the amount of low-viscosity modifier up to 90 ° C the dynamic viscosity was still increased. In particular temperature at 60 ° C stiffening of the bitumen will have a positive effect on the resistance to forming permanent deformation. It must be expected an increase the resilient modulus of elasticity. The fact of stiffening of the bitumen as well as an increase in viscosity will depend on the level of crystallization of paraffin in wax. The increase of stiffening of low-viscosity bitumen, on a smaller scale, is reproduced at 90 °C, which generates a conclusion that in bitumen there is a strong plate lattice of micro crystalline paraffins. At a temperature of 135 °C the situation was changing. This time the increase of content wax causes radically reduction the viscosity below 2 Pa•s, which was adopted as a necessary viscosity level of optimal covering aggregate. To make observation easer, values of dynamic viscosity were presented in a semi-logarithmic scale. The value of dynamic viscosity of 2 Pa•s was presented as the main axis of the domain. It can be concluded that the temperature of 135 °C is sufficient to ensure the optimum viscosity level of covering aggregate. It must be noted that the temperature of compaction could be decreased up to 30°C which is necessary to proper compaction of the asphalt concrete in the range of viscosity between 2 to 20 Pa•s [5]. The proper compaction of the asphalt concrete also affects a resilient modulus of asphalt concrete.

3. Methodology and Test Results Analysis of Resilient Modulus of Asphalt Concrete with Low-Viscosity Modifier

The experiment was carried out on asphalt concrete AC 11 S which was designed for wearing course layer in accordance with PN EN 13108-1. The reference asphalt concrete which constrained the principal aggregate of quartzite in amount of at 55% was designed. A dolomite mix 0/4 was in the amount of 24% and granite fractured sand were used to increase fraction contents. The modification of asphalt synthetic wax was made in the amounts of 1.5%, 2.0%, 2.5%, 3.0%, 3.5%, 4.0% by weight, in proportion to 35/50 virgin bitumen [3].

During determining process these parameters, an important element of research was to evaluate the homogeneity of the work. The study allows only a sample, in which the voids in the group was ranged outliner results in line with the Grubbs test [6][7].

The experiment was realized according to the research program within the confines of two factorial design of experiment. Fitted statistically significant model was a polynomial of second degree [8] [9]. As the fixed factors were set amount o low-viscosity modifier (L_V) and operation temperature (TEMP). Void fraction content (Wp) was a quantitative and random factor. The visual characteristic of variation of resilient modulus of elasticity, as a regression model, were presented in figure 2.



Wp =3,1 % (R^2=0,89) Wp=4,1 % (R^2=0,89) Wp=4,9 % (R^2=0,89)

Fig. 2. Resilient modulus of elasticity versus operational temperature and low-viscosity modifier.

It should be noted that a amount of low-viscosity modifier had a statistically significant influence on the resilient modulus of elasticity(p-value = 0.0215), as the amount of low-viscosity modifier increased. But the greatest changes in the level the resilient modulus of elasticity was observed during changing the operate temperature (p-value <0.001). This situation is related to the level of crystallization of aliphatic modifier, which in turn can lead to excessive stiffening of asphalt concrete. The void fraction content had a relevant influence. The highest resilient modulus of elasticity values were registered when the level of void fraction content was in the range between 3.0 and 3.5%. The constant increase in void fraction content, revealed as a lack of proper compaction, resulted in lowering the resilient modulus of elasticity especially at 25 °C. This situation may affect the ability of constant compaction of asphalt concrete and it is the beginning of rut forming process at higher temperatures. Additionally the excess void fraction content causes the marginalization of the effect modifier, as can be seen for void fraction content of 4.9%. Thus, the impact of the lack of rigor in the compaction process eliminates the benefits of using synthetic wax.

Resilient modulus of elasticity depends on the time of loading and temperature measurement. To further analyze of test results was a assessment of the thermal susceptibility of the resilient modulus of elasticity of asphalt concrete with asphalt low-viscosity modifier. The study was conducted for the Marshall samples in temperature ranges of 5 °C, 12 °C and 25 °C, all of them have been compacted at the optimum temperature of 125 °C [5]. Temperature of 12 °C is accepted as equivalent in Poland standard and it is a level of visco-elastic characteristics are accepted to the mechanistic designing method. The results of the dynamics of the expected and predicted value of resilient modulus of elasticity in term of the temperature, (performed in SAS) was presented in Figure 3.

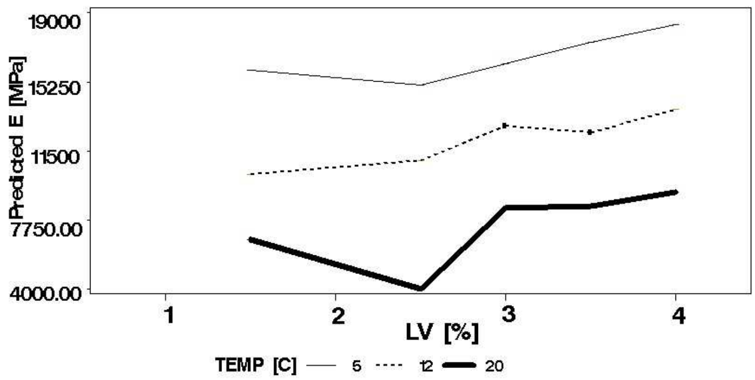


Fig. 3. Predicted value of resilient modulus of elasticity versus content of low-viscosity modifier (L-V) and operational temperature (TEMP).

Analyzing of the growth rate of the stiffness it should be noted that the proportional growth rate in relation to the level of the measurement temperature. The slope angle of trend growth of the x-axis is similar for different temperatures without impact of the dosage of the low-viscosity modifier. It can be concluded that a low dosage of the modifier in scope of 4% did not cause a sudden increase in stiffening the asphalt compared to other variants of dosage. Thermal susceptibility of low-viscosity bitumen with different variants of dosage of synthetic wax will not cause an excessive increase in the complex viscosity of asphalt concrete. The level of resilient modulus of elasticity will be proportionally vary depending on the amount of the modifier concentration.

4. Conclusion

Basing on the analysis of the test results of asphalt concrete the following conclusions can be drawn:

- the increase in the amount of synthetic wax causes the stiffening of the bitumen at operating temperatures below 60 °C and significant liquidates at 135 °C.
- the measurement of both temperature and amount of the low-viscosity modifier significantly affect changing in the resilient modulus of elasticity. In relation to the dynamics, the temperature measurement had the greatest influence,
- excessive voids fraction content significantly limited a beneficial stiffening effect of the modifier,
- thermal susceptibility of the asphalt concrete with the low-viscosity modifier remains at a similar level in the modifier dosage up to 4%.

References

[1] STEFAŃCZYK B., MIECZKOWSKI P., *"Mineral mix asphalt"* WKiŁ Warsaw 2008.

[2] PN-EN 12697-26:2004 *Hot mix asphalt - requirements* - Test methods for hot mix asphalt: Stiffness

[3] Product Information *The Bitumen Additive for Highly Stable Easy Compactible Asphalts* 2008.

[4] GAWEŁ, I.; KALABIŃSKA, M.; PIŁAT, J.. Asfalt drogowy [Road bitumen], WKŁ, Warsaw, 2001.

[5] IWAŃSKI M., MAZUREK G., *"Asphalt concrete of low-viscosity modifier"*, 2nd International Conference on Transport Infrastructures in Sao Paulo, Brazil, August 2010.

[6] GRUBBS, F. E., *Procedures for detecting outlying observation in samples, Technometrics*, Vol. 11, No 1, p 1-21, February 1969.

[7] *Guide to Expression Uncertainty in Measurement*, ISO Switzerland 1995.

[8] PIASTA, Z., LENARCIK, A.,: Methods of statistical multi-criteria optimisation, [in:] A. M. Brandt (red.), *Optimization Methods for Material Design of Cementbased Composites, E & FN Spon*, London, New York, p. 45-59, 1998.

[9] Montgomery, D. G. *Design and Analysis of Experiments*, 5th Edition, John Wiley and Sons, 2001.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Roundabouts for Thailand. Recommendations with a European View

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Abstract. European and Thai universities established an international network in order to improve road traffic safety in Thailand. In the framework of the common project a guideline for the design of roundabouts has been prepared. Site surveys were meant to get information about existing Thai roundabouts and traffic behaviour in order to fit the guideline to the local conditions.

Keywords: Roundabout, Traffic safety, Guideline, Site survey, Thailand.

1. Introduction

About 10 000 people died and 1 million people were injured in traffic accidents in Thailand per year in the last decade [8]. In the framework of an EU financed project titled "Improving Road Traffic Safety in Thailand - A Common Challenge for European and Thai Universities" five universities from Europe and Asia established an international network (Bauhaus-University Weimar, Germany; Szechenyi Istvan University, Gyoer, Hungary; Asian Institute of Technology, Thammasat University, Bangkok, Prince of Songkla University, Hat Yai, Thailand). From 2009 to 2010 a new design guideline was being prepared by the participating institutes for the design of roundabouts.

2. Site Surveys

As a part of this work, site surveys were made by the authors at existing roundabouts in order to ensure that the guideline is appropriate for the actual conditions in Thailand. The goals of the site surveys were to identify the local design solutions, to get acquainted with the local traffic behaviour and to get data on the typical traffic volumes, mix, speeds, etc.

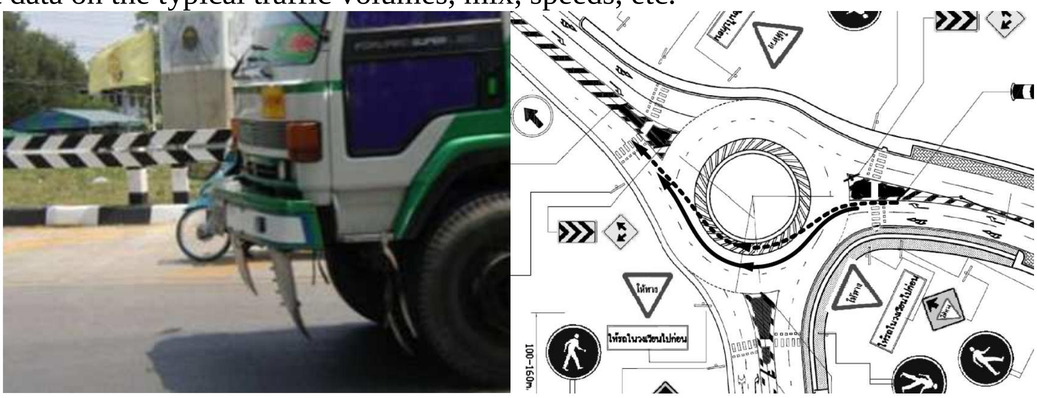

Fig. 1. Motorcycle overtakes a lorry on the painted truck apron .

The 8 sites selected were meant to represent the variety of Thai roundabouts. They are different in their geometric parameters, traffic conditions and location in the traffic network.

Figure 1 and Figure 2 show the different types of conflict situations and behaviour of drivers and pedestrians observed and registered by the authors [5].

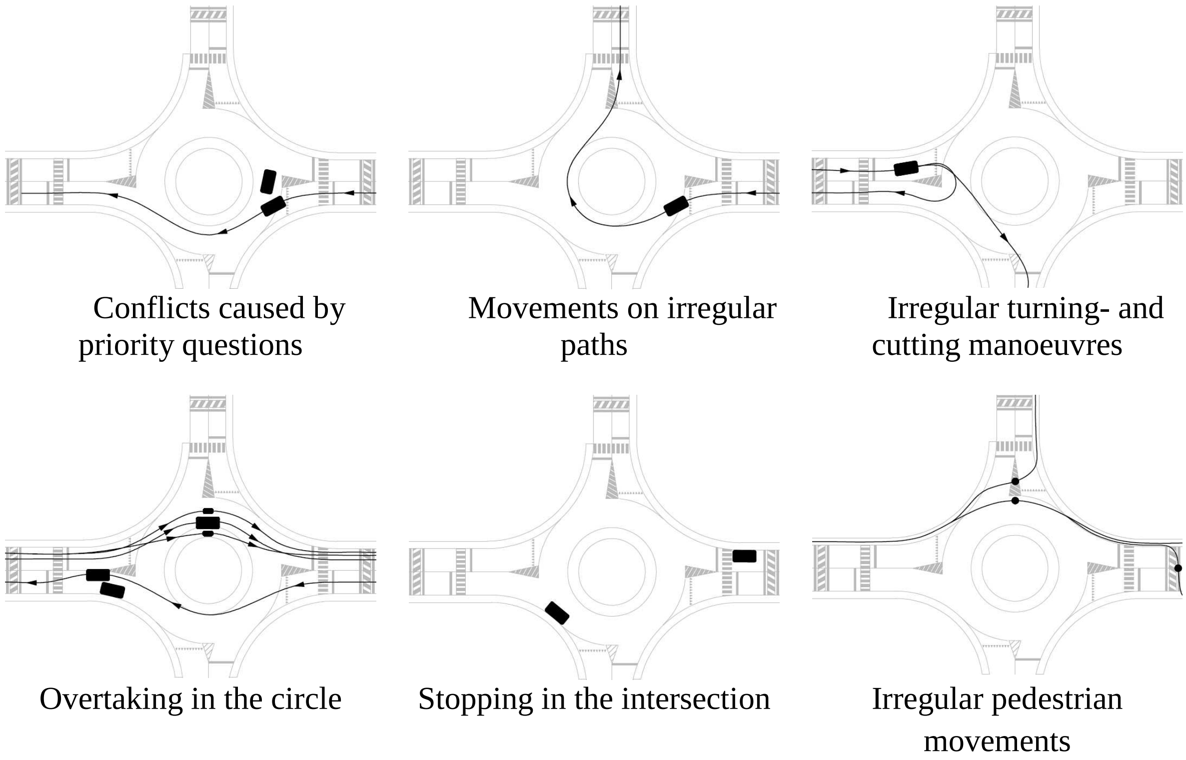

Fig. 2. Typical conflict situations and irregular manoeuvres at Thai roundabouts.

The ratio of irregular manoeuvres was relatively high, i.e. about 8%. From the observations, some general problems concerning irregular traffic manoeuvres and conflict situations were identified (Fig. 2.). It was found that the origins of these problems are in several cases in the design of the investigated roundabouts as listed below [6].

- The alignments of the intersection legs allow vehicles a higher speed.
- Splitter islands are often missing or they are not properly built (Fig. 3.).
- The circular roadway is built too wide; vehicles overtake in the circle (Fig. 1.).
- There is often too much available space at the entries and exits.
- In many cases there are no designated pedestrian crossings.

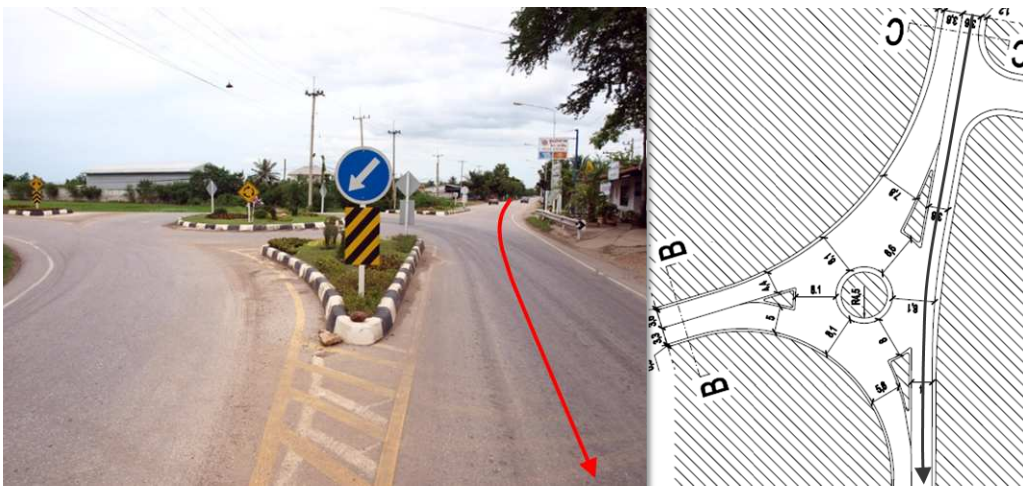

Fig. 3. Improper alignment of the intersection legs leads to high speeds.

3. Guideline for the Design of Roundabouts

The guideline uses the experiences from Germany [1, 2], Hungary [3] and Thailand [7]. The structure of the design guideline is as follows:

- General principles
- Areas of application
- Principal characteristics of roundabouts
- Roundabout types
- Geometric design of roundabouts
  - General requirements on the design of roundabouts
  - Elements of a roundabout
  - Central island, circulating roadway and truck apron
  - Entry and exit design
  - Splitter islands
  - Bypasses
  - Check of drivability, visibility
  - Pedestrians, public transport
- Traffic signs and road markings
- Capacity analysis
- Annex I. Safety checklist
- Annex II. Typical complete roundabout designs (12 design patterns, Fig. 5.)

The most important principles proposed by the guideline for the design of roundabouts corresponding to the observed problems [4]:

- Slow down vehicles by geometry: Placement size and shape of the intersection elements suggest the right speed level.
- The splitter islands should be correctly built: Recommended splitter island are the parallel, and the triangle shaped. Painted splitter islands are not recommended (Fig. 4.).
- Reduction of the width of the circular roadway: The guideline does not deal with multilane roundabouts. The ratio of the diameter and the circle width has been determined. Truck apron should be implemented with an elevated surface meant to separate from the circulating roadway.
- Reduction of the available space at the entries and exits.
- To build safe crossing for the pedestrians. Implementation of pedestrian crossings on the splitter island. The conclusion exemplifies the main results and the fundamental ideas presented in the manuscript.

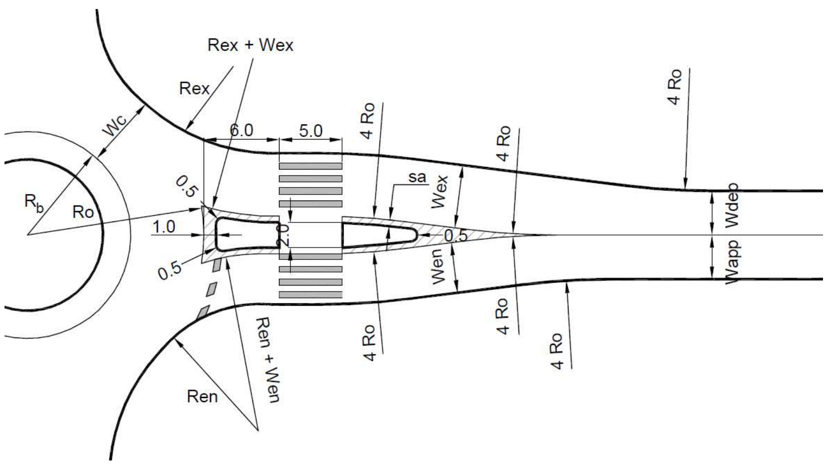

Fig. 4. The design of the splitter island with pedestrian crossing.

The design guideline for the "Design of Roundabouts" will foster the implementation of roundabouts in Thailand in the future. The guideline will provide road engineers necessary and useful knowledge about designing and constructing safe and efficient roundabouts in Thailand.

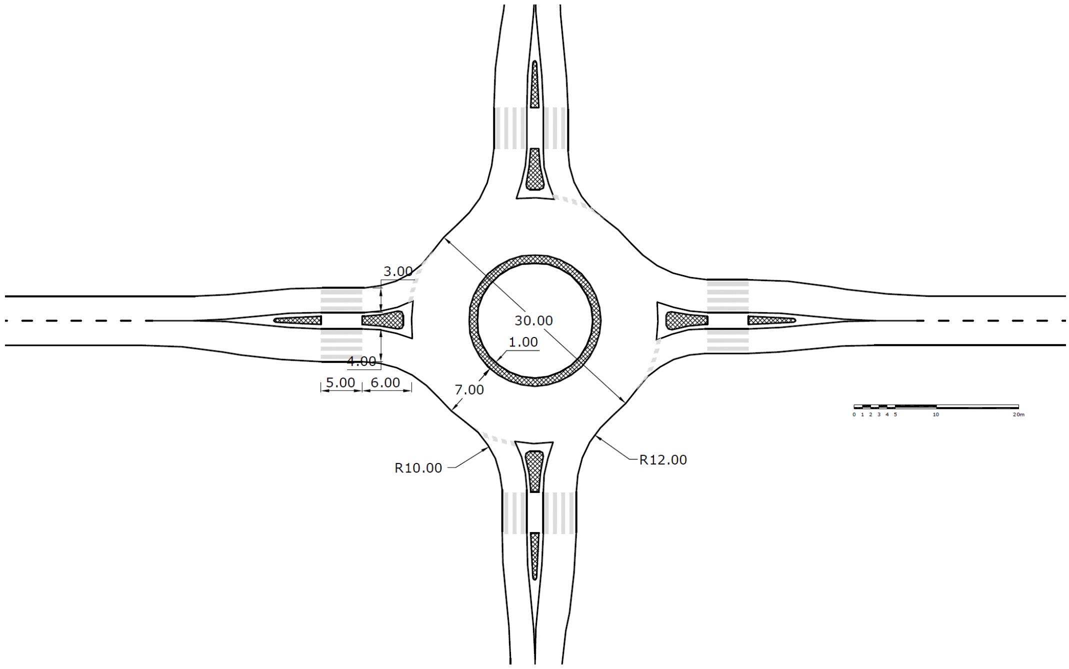

Fig. 5. Example of a recommended compact roundabout inside built-up area.

Acknowledgement

The preparation of this paper was supported by the EU-Grant Contract – External Aid- No. DCI-ASIE/2008/169-162) and TAMOP-4.2.1/B-09/1/KONV- 2010-0003: Mobility and Environment supported by the European Union and co-financed by the European Regional Development Fund.

References

[1] BILON, W., SCHNABEL, W. *Bewertung der Qualität des Verkehrsablaufs auf Hauptverkehrsstraßen, Straßenverkehrstechnik*, Kirschbaum Verlag, Bonn, 2003.

[2] FGSV, *Merkblatt für die Anlage von Kreisverkehren (German guideline).* Forschungsgesellschaft für Strassen- und Verkehrswesen, 2006.

[3] HRS, *Design of Roundabouts. Guideline No. ÚT 2-1.206/2001*. Hungarian Road Society – Ministry of Economy and Transport, 2001.

[4] KOREN, Cs. et al., *'Roundabouts – Preparation of design guideline for Thailand'*, TSTS Conference 2010, Bangkok, Thailand, 2010.

[5] KOREN, Cs, PESTI, B., VESPER, A., TANEERANANON, P., KANITPONG, K., IAMTRAKUL, P., *Roundabouts – preparation of a design guideline for Thailand*, First International Conference of Thai Society for Transportation & Traffic Studies. January 22-23, 2010. p. 1-16., 2010.

[6] MILETICS, D., PESTI, B.: *'Report on site surveys of roundabouts in Thailand'*, Unpublished interim report of the Thai-EC Project Improving Road Traffic Safety in Thailand. Szechenyi Istvan University, Gyor, Hungary, 2009

[7] OTP, *Traffic Signs Standard and Manual,* Office of Transport and Traffic Policy and Planning, 2003.

[8] TANABORIBOON Y., SATIENNAM, T., *'Traffic accidents in Thailand'*, Iatss research Vol.29 No.1, 2005, pp 88 – 100., 2005.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Preparation of Real Scale Samples of a Flexible Road Pavement for a Shear Test

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Abstract. The laboratory investigations are a worldwide methodology to examine research issues in several fields and it is a realistic approach, useful to collect reliable results. In particular, this work is focused on road pavement analyses and the aim of this work was to identify a methodology to build up real scale samples able to reproduce a multilayer flexible pavement. At the University of Parma, multilayer slabs were tested to obtain useful information related to the performance of the flexible roadway in asphalt concretes and during the in-service life. The investigations tested the stress/strain responses of the samples and for this reason it was necessary to insert proper strain gauges between layers. Hence, the specimens were built in layers and then fixed together, trying to maintain the slabs as in-continuum model. Consequently, a shear test was design to set and control the interface characteristics.

Keywords: Shear test, road pavement, laboratory investigation.

1. Introduction

At the laboratory of the University of Parma several studies were conducted on multilayer flexible pavements to investigate the road pavement during the in service life. For this reason, a straightforward methodology was defined, in order to prepare real scale samples that could perform as an in-situ road flexible pavement of asphalt concrete mixtures during the laboratory section. A suitable procedure to assembly the surface layers and the base course was designed to guarantee proper adhesion and connection during the testing phase and in order to avoid any debonding phenomenon. In fact, the investigated slabs were created in two phases: the superficial strata (wearing course and binder layer) and the base stratum. This procedure was necessary since strain gauges were positioned between binder course and base layer to investigate the strain responses of the samples under a real condition loading section. Subsequently, the two parts were connected and the “fixing procedure” was set running a proper shear test, in order to control the behavior at the interface and to obtain a proper adhesion. However, a preliminary bibliography research showed that this is a widespread topic studied in several scientific investigations, to examine the crucial point of the interactions between layers in road infrastructures. A useful report created by the Illinois Center for Transportation in 2008 is an extensive survey of the most important studies focused on this topic [1] in these last years, besides being a reliable laboratory investigation. Therefore, this report laid the bases for the shear investigations conducted in the laboratory with proper samples.

2. Laboratory Investigations and Results

The laboratory investigations were run on cylindrical samples which could study the interaction between the binder layer and the base course and their behaviors. The specimens were created with asphalt concrete specifically mixed in the laboratory and using alike limestone aggregates from Northern Italy and the same natural binder (PG 64-28). However the particle-size curves of the two asphalt concretes were different (base course 0÷30 mm and binder layer 0÷15 mm), but the concretes were mixed using a proper asphalt mixer available at the University of Parma. These two materials were used to prepare cylindrical samples (150 mm diameter) and around 60 mm height with a Gyratory Compactor and the equipment was set at 600 kPa and 100 gyrations. Following this procedure half of samples were realized using base mixture and half the binder asphalt concrete. This test did not require any specific volumetric properties, since during the shear test the interface behaviour was investigated and the most important aspect of the cylinder was the smoothness of the surface, the asphalt mixtures used and the aggregate sizes. Those specimens would have just recreated the surfaces to be put in contact in the real scale samples.

The two kinds of cylinders were put inside the two halves of a non-deformable steel box, dipped in cement mortar, but with the interface area clear which was checkable while the tests were carried out. Natural binder (PG 64-28) was used to put the surfaces in contact with an amount rate of 0.92 l/m^2 and was spread at the temperature of 150°C on both surfaces in contact. Moreover, the tests were carried out using a Material Test System machine (MTS) setting the displacement rate at 0.042 mm/s [2] and the pressure normal to the interface shear zone at 0.002 MPa. This value was calculated considering the weight of the asphalt material that rested on the interface zone of the real scale samples (the multilayer slabs), which is represented by the two surface layers (wearing and binder course). These two strata were 70 mm thick and the weight is around 4.2 kN, considering a slab of 500 mm x 500 mm. This meant that on the cylinders used for the shear tests, 0.35 kN were applied using loading cells (Fig. 1).

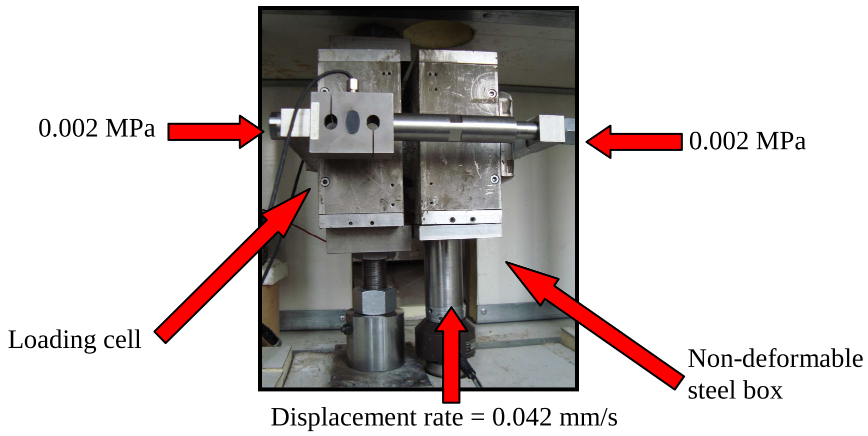

Fig. 1. Set-up configuration of the shear test.

One more important aspect of the test set-up was related to the pressure applied to make perfectly in contact the two halves of the cylinder inserted into the steel box, besides the time of the application of this force before the execution of the shear tests. The pressure of 600 kPa of the Gyratory Compactor was assumed as a reference, which was the same applied during the creation of the cylinders for these shear tests. In summary, it means that the cylinders prepared with that Gyratory Compactor were dipped into the cement mortar inside the steel box, and then hot natural bitumen at 150°C was spread on the two surfaces which were put in contact. Finally the proper adhesion was recreated applying around 10÷11 kN with the two load cells. In few minutes (around 3) the load cells reached the maximum value of load and after half an hour the load cells were applying a load approximately equal to zero. The test was carried out when the bitumen reached the room temperature and the sample could be considered completely adherent. This “fixing procedure” was executed at room temperature (around 15÷20°C).

Finally, it was fundamental to check if the load applied on the steel box with the two asphalt cylinders put inside would have caused permanent deformations. In that case the samples and the test itself would have been unreliable. Therefore, a new “test” was set to investigate the performance of the three different asphalt mixtures. Hence, on cylindrical samples a constant displacement rate was applied, in order to check the behaviour of the material before and post the failure range. For this purpose, cores 100 mm diameter taken from mono-material slabs were used.

Those cylindrical specimens could guarantee the same performance of the real scale slabs, since they had equivalent features (composition of the mixtures and compaction method, as well as volumetric and mechanical characteristics). Hence, this test was again carried out using the MTS machine and with the displacement control set at 0.084 mm/s until the complete failure of the cylinders. This further investigation was carried out at the room temperature (around 15÷20°C), the same set during the "fixing procedure". Fig. 2a shows few imagines during this investigation. Moreover, in Fig. 2b the load-displacement curves are shown for each asphalt mixture (the wearing, binder and base respectively).

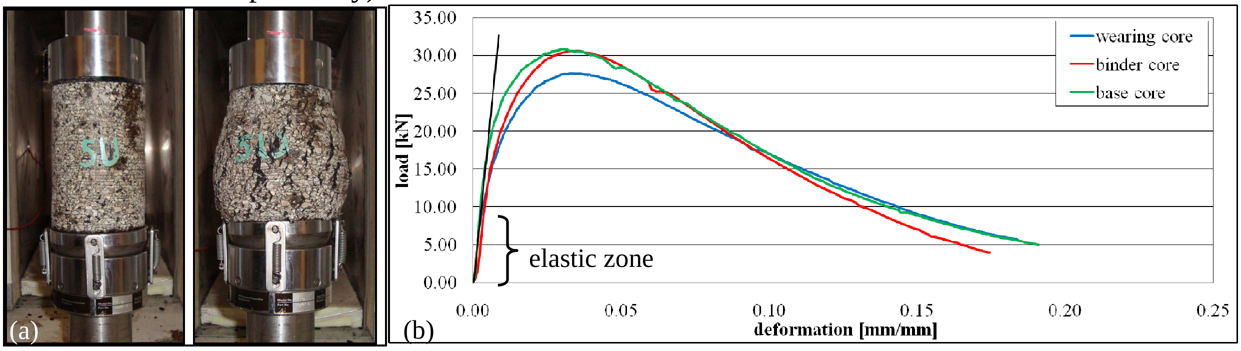

Fig. 2. Cylindrical samples tested to investigate the elastic behavior of the asphalt mixtures (a) and. elastic zone of the wearing course, binder layer and base course mixture (b).

The graph in the last Fig. shows the ranges of the elastic zones in the three different mixtures. The analysis of those curves highlighted that the elastic range was minimum for the wearing asphalt mixture core, which was around 7÷8 kN. Applying this force on a 100 mm diameter cylinder, it can be transformed into a pressure of 900÷1000 kPa. Therefore, the asphalt mixtures during the "fixing procedure" of the shear tests could be considered in a range of fully elasticity, since the load applied on the shear box (600 kPa) was 30÷40% less than the minimum value highlighted in the previous Fig. After this last control, it was possible to state that the set-up of the shear test was trustworthy.

After these fundamental steps, several shear tests were carried out following the procedure explained above and at the temperature of 10°C, in order to keep the asphalt mixture in a visco-elastic domain [3] and avoid any plastic damage. Fig. 3 shows an average of the results collected during the shear tests: the interface shear strength in MPa versus displacement in mm. The peak value is around 0.60 MPa.

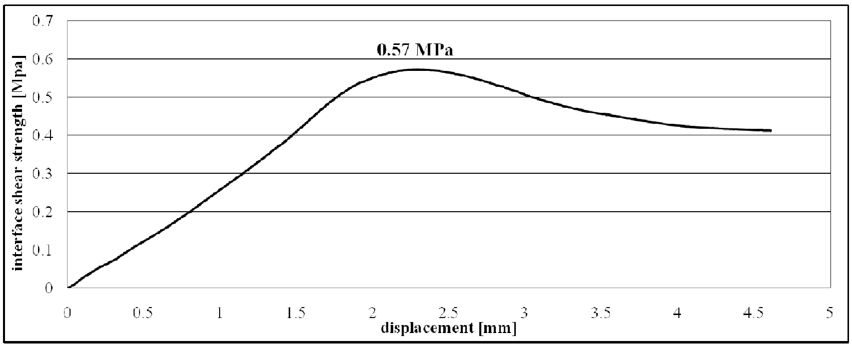

Fig. 3. Results collected during shear tests and the maximum interface shear strength.

3. Conclusion

Is it possible to state that this is suitable interface shear strength according to the purpose of this work? In fact, the aim of this shear test was to guarantee a proper adhesion between the binder layer and the base course which was created separately and connected during the carrying out phase of the real scale slabs during the laboratory section. As mentioned in the introduction, this consisted on applying a certain pressure which can reproduce the traffic load on a multilayer flexible pavement fixing proper strain gauges at the interface. Consequently, in order to answer to the question above, a further investigation became necessary. Therefore, the software BISAR was used to reproduce the same multilayer pavements created in the laboratory and simulating the passing of a truck wheel. In that way it was possible to check the horizontal stress level at the interface between the binder layer and the base course. This software considers a semi-indefinite multilayer system, where the asphalt mixtures are isotropic linear elastic. In Fig. 4 the Block Report is shown and it summarizes the results coming from the simulation of a truck wheel with the software BISAR. The horizontal stress between the binder layer and the base course is around 16 times less than the peak interface shear strength collected with the shear tests in the laboratory. In fact, it is possible to check the value of the horizontal shear stress between the binder layer and the base course calculated using BISAR (0.03495 MPa) and to compare it with the same datum get from the laboratory investigations (0.57 MPa). Therefore, even if BISAR makes a preliminary assumption of linear elasticity and the laboratory shear tests considered the material as a visco-elastic mixture, this considerable difference between the two values could make the interface shear strength reliable and the two upper surfaces could be considered an in-continuum system with the base layer.

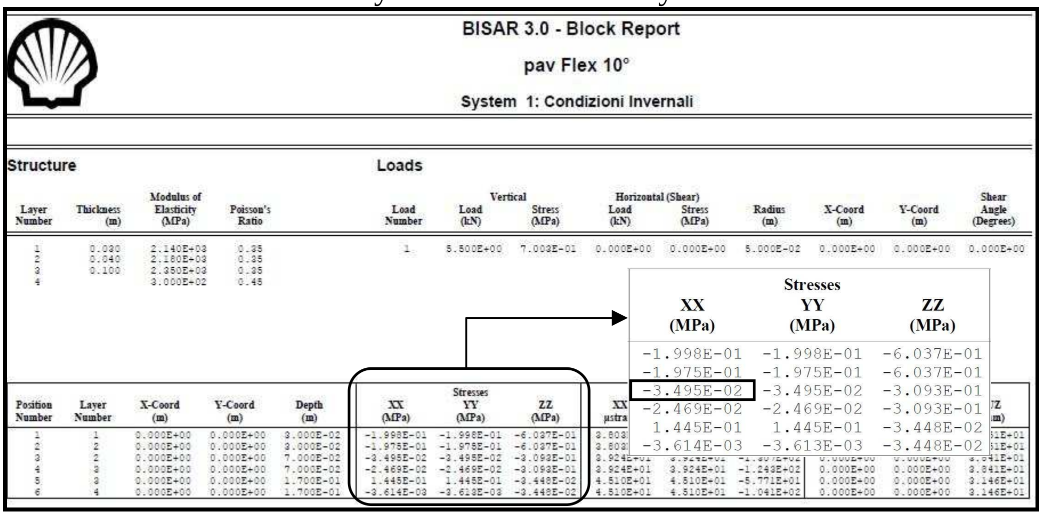

Fig. 4. Bisar report with the horizontal stress between base and binder layer highlighted.

Consequently, it was possible to state the method set to assembly the slabs for a laboratory section was consistent and it permitted to say the multilayer samples could work in continuum. In this way it would be possible to insert equipments to collect data between layers creating no discontinuities.

Acknowledgement

The authors would like to sincerely thank Dr. Arianna Costa for her collaboration and support in starting this research.

References

[1] AL-QADI I. L., CARPENTER S. H., LENG Z., OZER H., TREPANIER J. S. *Tack Coat Optimization for HMA Overlay*. Research Report FHWA-ICT-08-023, Illinois Center for Transportation, USA, 2008.

[2] SANTAGATA F. A., PARTL M. N., FERROTTI G., CANESTRARI F., FLISCH A. *Layer Characteristics Affecting Interlayer Shear Resistance in Flexible Pavements.* Journal of the Association of Asphalt Paving Technologists, Vol. 77, pp 221-256, (2008).

[3] READ J. M., COLLOP A. C. *Practical Fatigue Characterization of Bituminous Paving Mixtures*. Journal of the Association of Asphalt Paving Technologists, Vol. 66, pp 74-108, (1997).

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

The Use of Foamed Bitumen for Soil Stabilisation

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Abstract. Foamed bitumen is a new material used for soil stabilisation. Its suitability was assessed by conducting laboratory stabilisation tests for uniform-grained soil. The contents of foamed bitumen were 3,0% and 3,5%, while the contents of portland cement added to increase the grain quantity were 2,0%, 2,5% and 3,0%.
The test results show that soil stabilised with foamed bitumen meets the same criteria as those established for soil stabilised with 6,0% cement and for soil stabilised with recycled base material obtained by applying the foamed bitumen and the M-C-E technology. It is reported that by applying foamed bitumen it is possible to produce a semi-rigid layer of stabilised soil.

Keywords: Stabilisation, foamed bitumen.

1. Introduction

The use of binders in the stabilisation process results in increased compressive strength of the stabilised soil and, consequently, increased rigidness and significant shrinking deformations, which usually lead to the formation of cracks across the stabilised layer. As a result of this process considerable cracking reflected onto the road surface occurs, which is the cause of a loss in its durability. In order to avoid this destructive process, bitumen binder can be used.

Bitumen is a viscoelastic material. During the soil stabilisation process it does not lead to an increase in rigidness and no cracking occurs throughout the operational time of the produced layer of the stabilised soil. Additionally, such a stabilised soil layer can be used as a supporting base of the bitumen type.

Soil stabilisation technologies have been developed that use liquid bitumen produced with solvents. However, this kind of bitumen binder is expensive and can have a negative impact on soil and water as a result of solvent penetration to the ground. The application of bitumen emulsion has also been limited for road building, since bitumen emulsion contains a lot of water – up to 45%, which results in elongating half-life and maintaining the required stabilised soil parameters in the long period of time.

Only the introduction of foamed bitumen to the road building technology enabled to use bitumen binder for soil stabilisation [1, 2]. The foamed bitumen technology was developed in Africa and Australia. That is the reason why tests to assess its effectiveness for local soil stabilisation under the Polish conditions are necessary.

2. The Aim and Scope of the Investigation

The aim of the project is to determine the impact of foamed bitumen on the properties of the stabilised soil. The scope covered the following two stages:

a) stage I: recognition of basic bitumen properties, determination of foam bitumen properties, determination of the optimal water content for bitumen foaming,

b) stage II: recognition of the kind of the granular soil, determination of characteristics of the bitumen binders, performing tests on soil stabilised with foamed bitumen, formulation of conclusions.

3. Material and Test Results

3.1. Material

For the tests of the impact of foamed bitumen uniform – grained soil was used, which is classified as hardly compactable soil.

Basing on the tested parameters listed in Tab. 1 and granulometric analysis this soil was classified as flour sand, which is hardly compactable soil.

No.	Parameter	Value			
1	Sand equivalent	19,7 %			
2	Organics content	None			
3	PH	8,0			
4	Optimal humidity	9,2 %			
5	Maximal density of the soil matrix	1,987 Mg/m^3			
6	Coefficient of graining non-uniformity	3,6			
7	Flour sand (basing on the areometric analysis)	PN-86-B-02480	f'_p = 68 ÷90%	f'_Π = 10 ÷30 %	f'_i = 0 ÷2%
		result	f'_p = 80,48 %	f'_Π = 18,02%	f'_i = 1,5 %

Tab. 1. Basic soil properties.

Despite the fact that sand equivalent was less than 20 %, this soil was classified as flour sand (no. 7 in Tab. 1) due to flour and loam fraction contents and basing on the Feret triangle according to PN-86-B-02480. It is an exceptional case.

In order to obtain compaction according to the standards, it is necessary to stabilise this kind of soil.

3.2. Foamed Bitumen Properties

The fundamental foaming criterion of bitumen is its ability to produce foam of certain quality requirements [3, 4, 5]. During the tests two basic foamed bitumen parameters were assessed:

- expansio ratio (WE),
- half-life of the foamed bitumen (t½).

The optimal water content, which ensured the required foaming parameters, was also determined (Tab. 2).

Kind of bitumen	Water content [%]	WE Expansion ratio		t½ Half - life [s]	
		Determined	Required	Determined	Required
O 50/70	2,5	6,16	15 - 20	8,14	10 - 15
O 70/100	2,0	9,84		11,41	
L 70/100	2,5	6,14		6,97	
EL 70/100E	2,5	8,53		7,14	
Nyfom 85	2,0	15,85		15,28	
Nyfom 190	2,0	15,11		10,39	

Tab. 2. Foamed bitumen parameters for the recommended water content for foaming.

According to the requirements [1], soil stabilisation should be conducted with bitumen of as little viscosity as possible to enable proper covering of fine soil particles. Consequently, Nyfoam 190 bitumen was used for soil stabilisation tests. Its foaming characteristics has been presented in Fig. 1.

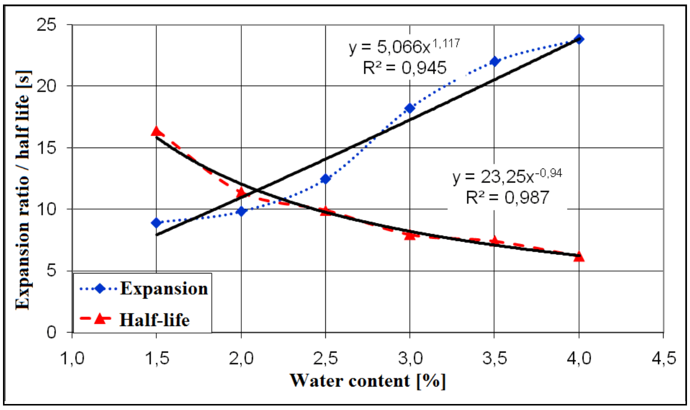


Fig. 1. Foaming characteristics of Nyfoam 190 bitumen.

3.3. Test Results of the Stabilised Soil

According to the requirements in PN-S-96012:1997 the effectiveness of foamed bitumen to improve soil parameters has been confirmed basing on the following characteristics:

- compressive strength after 28 days,
- frost resistance.

With the use of foamed bitumen as a stabiliser, the final compressive strength of the stabilised soil is achieved after 28 days, while there are no strength tests after 7 days.

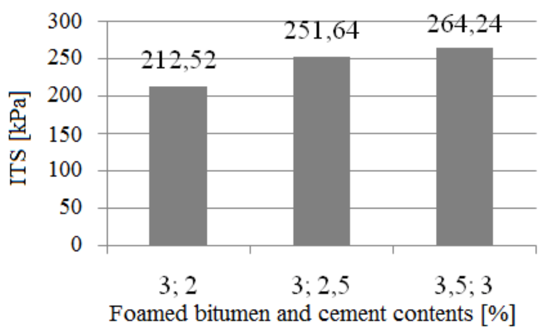
Cement was added to the mixes of soils stabilised with foamed bitumen in order to increase the fine fraction content in the ground and to improve tightness of the stabilised layer.

An important element of the tests was to determine the mechanical parameters of this material. The assessment was conducted basing on the requirements for recycled pavement in the M-M-C-E technology [6] and recycled pavement with the foamed bitumen technology [3].

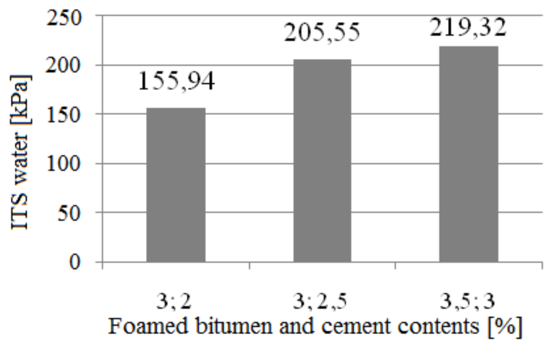
The test results of the impact of the Nyfoam 190 foamed bitumen on the properties of the stabilised soil have been presented in Tab. 3 and Fig. 2.

Material	ASp [%]	C [%]	S mean [kN]	ε mean [mm]	S / ε [kN/mm]	R28 mean [MPa]	WR frost
Soil Cement	3,0	2,0	3,92	1,58	2,48	2,65	0,81
	3,0	2,5	2,97	1,85	1,60	1,86	0,74
	3,5	3,0	2,54	1,81	1,48	1,37	0,67

Tab. 3. Impact of Nyfoam 190 bitumen on the stabilised soil properties.



a)



b)

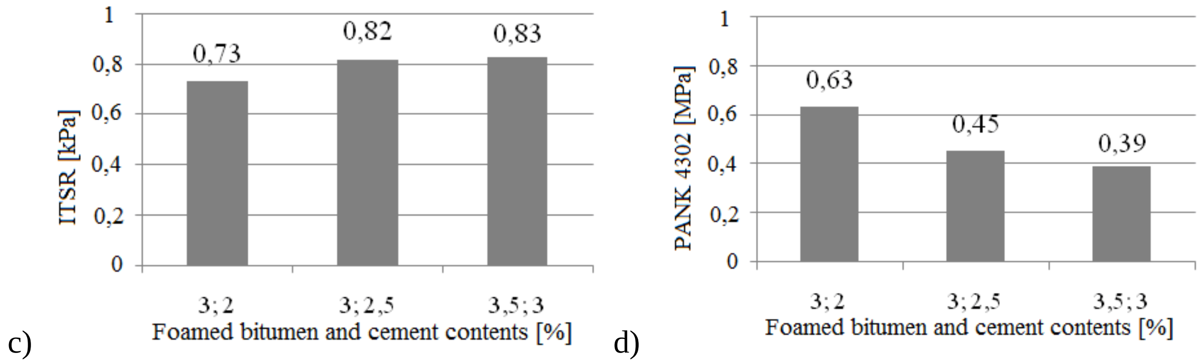


Fig. 2. Impact of the foamed bitumen and cement contents on: a) indirect tensile strength in the dry state ITS, b) indirect tensile strength after soaking with water ITS water, c) resistance to the effects of water ITSR, d) frost resistance according to PANK 4302.

The soil stabilised with the foamed bitumen content of 3,0 % and cement content of 2,0 % has most favourable properties. At these foamed bitumen and cement contents in the stabilised soil ITS meets the criteria as for the recycled pavement material in the foamed bitumen technology, for which ITS should be over 150 kPa. While ITSR is higher than 0,7, which also fulfills the minimal requirements with regard to the resistance to the effects of water.

Resistance to low temperature cracking of this soil is also ensured, since ITS < 4,8 MPa according to PANK 4302. Comparing the properties of the stabilised soil with the requirements as for the M-C-E mix it can be stated that the requirements with regard to Marshall stability and deformations are actually met – as for the road base loaded with KR 1-2 traffic.

4. Conclusions

Basing on the laboratory test results of the soil stabilised with foamed bitumen the following conclusions can be drawn:

- the use of 3,0% foamed bitumen and 2,0% cement contents ensures meeting the requirements according to the standards with regard to compressive strength and frost resistance of the stabilised soil,
- with the addition of foamed bitumen and cement to increase the fine fraction content, it is possible to create a semi – rigid stabilised soil layer, which not only has the proper compressive strength but also indirect tensile strenght ITS,
- the use of foamed bitumen to stabilise uniform – grained soil can create improved ground layer but also a surface base.

References

[1] Jenkins K.J., Collings D.C, Thesey H.L., Long F.M.: *Interim Technical Guideline: Design and Use of Foamed Bitumen Treated Materials*. Edited by Les Sampson of Asphalt Academy. ISBN 0-7988-7743-6. Asphalt Academy, Pretoria, South Africa, 2003.

[2] Polakis A., Alevra E.: *Pavement Rehabilitation by Recycling with Foamed Bitumen and Cement*. 2nd International Symposium Treatment and Recycling of Materials for Transport Infrastructure. Paris, 24-26th October 2005, C024

[3] *Wirtgen Cold Recycling Manual*. Wirtgen GmbH. Windhagen, Germany, 2004.

[4] Iwański M.: *Podbudowa z asfaltem spienionym.* Drogownictwo, 2006, 3, p. 97-106.

[5] Iwański M., Chomicz A.: *Przydatność do spienienia asfaltów drogowych stosowanych w Polsce*, Drogownictwo, 2006, 8,p. 267-271.

[6] *Zawadzki J., Matras J., Mechowski T., Sybilski D.: Warunki techniczne wykonywania warstw podbudowy z mieszanki mineralno-cementowo-emulsyjnej*. Zeszyt 61 IBDiM, Warszawa 1999.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

The Classification of the Testing Methods for Alkali-Carbonate Reaction

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Abstract. There are numerous methods of testing aggregate reactivity, most of which can be classified into a few basic groups, with regard to the object of research. With the knowledge on what particular methods are capable of, it is possible to plan the appropriate course of action. It is especially important when the reaction mechanisms are not exactly known, as in the case of the reaction of alkalis with carbonate aggregate (ACR). The properties of these aggregates make the application of testing methods different than those for silica aggregates not only advisable, but sometimes also necessary. The tests on the reactivity of silica aggregates provide the basis for the formulation of the majority of standards for AAR testing that are in force today. The objective of this paper is to present the methodology of ACR testing, taking into consideration the properties of this aggregate.

Keywords: Carbonate aggregate, Alkali-carbonate reaction, Testing methods.

1. Introduction

The alkali-carbonate reaction (ACR) is the second cause (after the alkali-silica reaction – ASR) of the inner concrete corrosion, resulting from the unfavourable interaction between cement and aggregate. It leads to the emergence of a swelling gel, causing the increase of the concrete volume, accompanied with the emergence of cracks decreasing its strength and increasing its absorbability. The mechanism of alkali-carbonate reaction is less known than the mechanism of the more common alkali-silica reaction. That is the reason why the majority of the testing methods of the alkali-aggregate reaction have been developed for the alkali-silica reaction. Furthermore, it is presently believed, as in the case of Katayama [1, 2], that the alkali-carbonate reaction is in fact a kind of alkali-silica reaction, and it is the cryptocrystalline quartz or opal present in the aggregate that is responsible for the expansion of the concrete with carbonate aggregate. According to other theories, the essential effect of the alkali-carbonate reaction is connected with the process of dedolomitization and the formation of brucite, or with the swelling of clayish minerals present in aggregate, or with the action of osmotic forces resulting from the activity of clayish minerals surrounding the carbonate grain as a semi-permeable membrane (it lets the water molecules through, but stops sodium and potassium ions) [3]. As there is a multitude of factors that could theoretically evoke the expansion of concrete with carbonate aggregate, and since the exact mechanisms of the process are unknown, the methods testing whether a particular aggregate is reactive should be very carefully selected. It is especially important in this case, since many tests aiming at the detection of an aggregate reactivity are of the long-term character.

2. Methods of Testing

In order to systematize numerous methods testing the alkali-aggregate reaction, they can be divided into three basic groups, with regard to their objectives. This division is presented as a diagram in Fig.1 In order to examine the reactive aggregate it is recommended that tests be conducted starting with group I and finishing with group III. Group I includes tests concerning the composition and properties of the aggregate itself. Among those tests one can distinguish chemical and petrographic tests aiming at the detection of potentially reactive phases in the aggregate and determining whether their number poses a threat for the concrete durability. The chemical tests are described i.a. in RILEM AAS-1 annex [4], or in ASTM C289-07 [2], or in the standard being in force in Poland PN-B-06714-47 [5]. Tests of this type may lead to the loss of the aggregate weight, caused by the fact that a part of its components pass into the NaOH solution as a result of the reaction with alkalis, taking place in a particular time. Similar tests are conducted also using HBF_4, or HCl in the case of carbonate aggregates [4]. Most standards expect that such tests should be conducted in a heightened temperature in order to shorten their duration. Examining dolomite rocks specified in the standard ASTM C586-05 [6], which is a certain modification of the above method, can also be included among the methods testing the aggregate properties. Instead of defining the change in weight, the method defines the length changes in the rock cylinder. The rock is designed as the concrete aggregate and is immersed in 1N NaOH solution in the room temperature.

The reactive components of aggregates can be also detected through their X-ray powder diffraction (XRD) or observations with the use of the optical microscope and through SEM combined with EDS analysis. One should however take into consideration the limitations of those methods.

Group I tests enable the detection of potentially reactive aggregate, however, the tests do not give a full picture of how a particular aggregate will behave when added to the concrete mixture. A short time required to do those tests is the advantage of this tests, therefore they are suitable for the initial detection of potentially reactive aggregate.

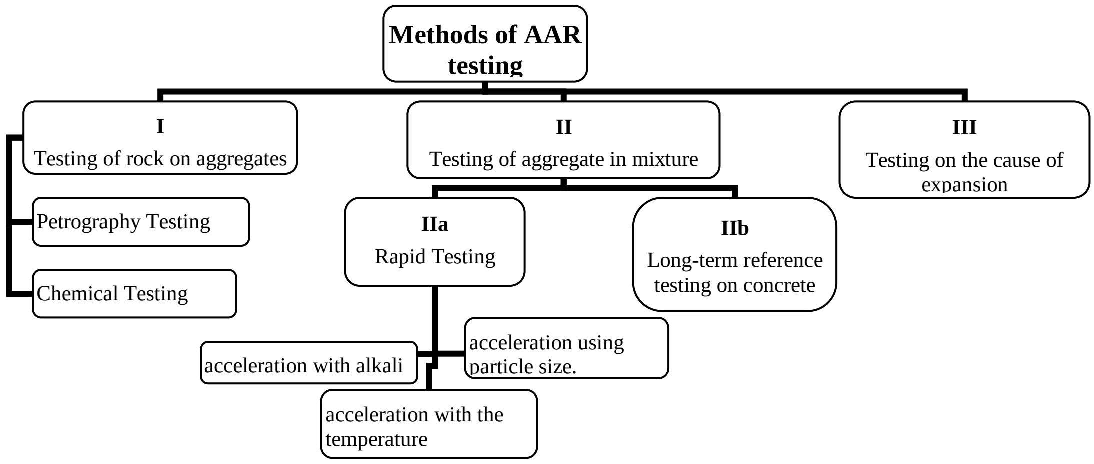

Fig. 1. Distribution methods of AAR testing.

It is only the group II tests that give the full answer to the question whether a particular aggregate will react with the alkalis in concrete. Those tests deal with the interaction of aggregates with the environment in which they will find themselves. This interaction results in the changes of the sample lengths measured e.g. with a Graf-Kaufman apparatus. During those tests one can also observe the development of cracks on the concrete surface, however, the analysis of the cracks formation is dealt with by group III tests. In this group one can distinguish two subgroups. The first subgroup includes accelerated tests, in which the mixture composition and the conditions in which the reaction takes place, are modified. The second subgroup includes tests on concrete samples, performed in the conditions comparable to the operational ones (ASTM C1293-08B [7], RILEM AAR-3 [8]). Due to the above reasons these tests take the longest time to be performed.

In order to accelerate the tests, the applied mixtures contain only the most reactive aggregate fractions (ASTM C227-10 C441-05, C1567-08 [7], RILEM AAR-2, 5 [8]), or alternatively the tests are conducted in heightened temperature (ASTM C1567-08 [7], RILEM AAS-2, 4, 5 [8]), or the amount of alkali in the system is increased (ASTM C1567-08 [7]). Frequently both modifications are used in order to shorten the testing time. Due to the way they are conducted these tests provide only the qualitative information on whether a particular aggregate is reactive. It is only the examination of concrete samples performed in ordinary conditions that can give the quantitative information on the behaviour of an aggregate in concrete.

Group III tests are designed to provide information on the cause of the concrete swelling, on the course of the reaction mechanism and on the changes in the concrete microstructure. In order to obtain this information, the most frequently conducted tests include X-ray powder diffraction (XRD) and SEM analysis combined with EDS analysis in micro-areas.

The research material for this investigation may come from the powdered samples from group II tests, but this means the investigation can only be conducted after the tests on mortar or concrete samples have been finished, and such tests may take many weeks to be performed. Sample investigations after petrographic research are also possible, however their limitations should be borne in mind. For that reason the samples designed exclusively for group III tests are also obtained. Separate bars are made and after being powdered they provide material solely for those tests. The investigation of samples cut out of the rock cylinder immersed in the cement paste permits a thorough examination of the changes in the composition and microstructure resulting from alkali-aggregate reaction taking place in different concrete micro-zones [9].

3. Testing of Carbonate Aggregates

The initial stage of the test on aggregate's susceptibility to their reaction with alkali, regardless of the kind of the aggregate, should be the petrographic test, as suggested by RILEM AAR-0 [8]. It is during those tests that it can be confirmed whether a particular aggregate is in fact carbonate and its reactivity can be initially defined (typically either the lack of reactivity or else a medium or high level of reactivity). In the chemical tests commonly applied for silica aggregate the content of the soaked grains is defined for the aggregate immersed in NaOH solution and the content of the silica dissolved in the solution is also defined.

Testing carbonate aggregates, obtained as a result of crushing massive rock, makes it possible to follow the changes in the linear dimensions of the samples cut out from the rock destined for aggregate. It is not possible in the case of silica aggregates (which may also appear as eg. gravel).

The ASTM C586-05 [6] standard is an example of such a test. The obtained value of the length difference has the advantage of being translated directly into the key effect of the alkali-aggregate reaction, i.e. the concrete expansion. It is especially important, as the exact mechanism resulting in this volume increase is unknown. Hence it is difficult to translate the weight changes of the carbonate aggregate subjected to NaOH into its reactivity. This method permits e.g. defining to what extent the aggregate expansion can be accumulated by the cement paste. In comparison to chemical tests measuring the weight loss, the C586-05 test takes a long time to be conducted (in ordinary conditions) and it is a disadvantage of this method.

While examining the composition of the carbonate aggregate using the XRD or SEM – EDS, the limitation of these methods should be borne in mind [2]. SEM– EDS testing may not detect tiny, dispersed quartz crystals contained in the aggregate. Detecting the presence of amorphous phases like opal may also cause difficulties, as they are nearly invisible in the X-ray diffractogram .

In the accelerated tests on the carbonate aggregate samples in order for the clearest results to be obtained in the shortest possible time, it is recommended that the coarser aggregate fractions be used (typically 4-10 mm) [10, 11]. It may result from the fact that a considerable part of the ACR reaction for the finer aggregate may already take place at an early hydration stage on its surface, whereas for the coarser aggregate it occurs later, when the concrete has hardened, since the alkalis must first enter into the inner structure of the aggregate with its reactive phases. The increased reactivity of these carbonate aggregate fractions can be explained (assuming that ACR = ASR) by the fact that most of the alkali gel emerging in the alkali-silicate reaction (ASR) around large grains remain unchanged through the process of carbonatization, which permits the expansion increase.

Furthermore, in contrast to small grains, in the large ones it is more difficult to limit the pressure exerted by the ASR gel filling the pores resulting from the dedolomitization [1].

The ACR acceleration is achieved by higher temperatures of the sample storage, as in the case of the ASR acceleration, where samples are usually stored at 80°C. Among the ACR tests there are also those conducted at higher temperatures 150 °C [11], i.e. in hydrothermal conditions, which can largely shorten the time required for the test to be performed and for the clearer results to be obtained. In these tests one should be concerned whether the processes correspond to those taking place at room temperature. The expansion acceleration is also facilitated by the fact that, with a defined alkali content in the system, the cements with lower alkali content are used in the tests and the missing amount of alkali is added with the molding water [11].

Examining the ACR course and the changes that it makes in the concrete microstructure is more complicated than in the case of ASR. It is caused by the fact that the mechanisms of this reaction are not yet fully known. It is necessary not only to investigate the emergence of the gel resulting from the ASR, but also to pay attention to changes connected with the dedolomitization process, i.e. the action of the osmotic forces [1, 3]. Nevertheless, carbonate aggregates make it possible to take rock samples, which can then be immersed in the cement paste, thus facilitating the investigation of appropriate zones of the system [9]. Such samples facilitate the selection of places for SEM observation.

4. Conclusion

Although testing carbonate aggregate reactivity is based on the same principles as testing silicate aggregate, ACR has its own special character and therefore its testing should be conducted in a modified way. In the case of carbonate aggregate the research can be conducted on rock segments and at higher temperature. Since the ACR mechanism is not yet fully known, several testing methods should be applied. It is a matter of great importance that the obtained tests results are as accurate as possible. It should also be remembered that the X-ray examination of the aggregate, while enabling the identification of its many components, is not a perfect method as it does not detect the reactive silica.

References

[1] KATAYAMA, T. *Materials Characterization* 53 (2004) 85-104: How to Identify carbonate rock reaction in concrete.

[2] GRATTAN-BELLEW, P. E., MITCHELL, L. D., MERGESON, J., MIN, D. *Cement and Concrete Reseach* 40 (2010) 556-562: Is Alkali-Carbonate Reaction Just a Variant of Alkali-Silica Reaction ACR = ASR.

[3] KURDOWSKI, W. *Chemia cementu i Betonu.* SPC, Kraków, 2010.

[4] SIMS, I., NIXON, P. *Materials and Strctures* 36 (2003) 480-496: RILEM recommended Test Metod AAR-1: Detection of Potential Alkali-Reactivity Potential of Aggregates – Petrographic method.

[5] PN-B-06714-47:1988 *Mineral aggregates. Testing. Determination of potential alkali reactivity. Determination of soluble silica in sodium hydroxide content.*

[6] ASTM C586–05 *Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks as Concrete Aggregates*

[7] THOMAS M. et AL. *Cement and Concrete Reseach* 36 (2006) 1842-1856: The Test Methods for Evaluating Preventive for Controlling Expansion Due to Alkali-Silica Reaction in Concrete.

[8] SIMS, I., NIXON, P. *Materials and Strctures* 36 (2003) 472-479: RILEM recommended Test Metod AAR-0: Detection of Alkali-Reactivity Potential in Concrete – Outline guide to the use of RILEM methods In assessment of aggregates for potential alkali-reactivity.

[9] TONG, L., TANG, M. *Cement and Concrete Reseach* 21 (1999) 361-373: Expansion Mechanism of Alkali-Dolomite and Alkali-Magnesite Reaction.

[10] SOMMER, H., SIMS, I., NIXON, P. *Materials and Strctures* 38 (2005): AAR-5 787-792: Rapid Preliminary screening test for carbonate aggregates.

[11] XU, Z., LAN, X., DENG, M., TANG., M. *Cement and Concrete Reseach* 32 (2002) 851-857: A New Accelerated Method for Determining the Potential Alkali-Carbonate Reactivity.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

The Evaluation of the Laboratory Methods Applied in the Identification of the Alkali-Silica Reaction

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Abstract. The alkali-silica reaction is one of those reactions that cause the concrete destruction. The question of the alkali-aggregate reaction has not been fully explained yet. This paper presents the outline of the reaction mechanism, and discusses the most common methods of investigating the reactivity of aggregates and of aggregate-cement mixtures. The ideal testing method should be fast, simple, precise and accurate. None of the accessible methods combine all those assumptions. The paper presents the evaluation of the applied methods.

Keywords: Alkali-silica reaction, Aggregate, Testing methods.

1. Introduction

The expansion processes resulting from the alkali-aggregate reaction lead to both the utalitarian and the aesthetic values of the buildings being decreased The reaction is slow, therefore its destructive effects become evident only after a longer period of time. The progress of the alkali-aggregate reaction depends on many factors, i.a. on the composition of concrete pore solution and on the presence of reactive aggregate.

Thus the evaluation of the aggregate reactivity appears to be necessary before its application in real life structures. The probability of the reaction can be determined by direct analysis tests performed on aggregates and by indirect analysis of the standard mortar and concrete samples.

2. Mechanism of Alkali-Silica Reaction [1]

The alkali-silica reaction takes place between reactive components of silica aggregates and alkalis present in cement. In order for the reaction to take place there must be favourable conditions: high moisture, high pH and the reactive silica. The reactive silica reacts with hydroxyl groups present in the concrete pore solution. The effect of sodium and potassium ions on the silica grain results in the destruction of the three-dimensional structure of quartz. This process gives polymineral gel, which is capable of absorbing water and thus of giving an expansive product. The resultant gel is surrounded by mortar components and therefore an osmotic pressure is generated. This pressure exceeds the tensile strength of concrete, which consequently leads to concrete cracking. As the water content grows, the resultant gel can move in concrete, filling pores and cracks.

3. Methods of Testing the Occurrence of the Alkali-Silica Reaction

Petrographic and chemical methods can be distinguished among the most common methods evaluating the alkali-aggregate reactivity, both according to American standards ASTM and European standards RILEM.

The testing of the alkali-aggregate reactivity is performed through:

a) aggregate petrographic analysis
b) mortar and concrete samples analysis in ultra-fast laboratory tests
c) mortar and concrete samples analysis in long-term tests

3.1. Aggregate Petrographic Analysis

American method ASTM C295 [2] uses the following petrographic techniques to evaluate siliceous aggregate reactivity: optical microscopy, energy-dispersive X-ray spectroscopy (EDS) and X-ray powder diffraction (XRD**)**, infrared spectroscopy, and thermic analysis . The analysis is performed on samples in the form of core samples, cross-sectional samples or pieces of aggregate. The analysis of the obtained picture permits primarily the qualitative determination of minerals (optical microscopy), and their quantitative determination, detection of voids, defining the size and shape of grains (computer analysis of the picture).

As far as silica aggregates are concerned , the reactive minerals include: opal, cristobalite, tridymite, obsidian, siliceous glass, horn-stone, chalcedony, cryptocrystalline volcanic rocks i.e. andesite, rhyolith, metamorphic quartz under stress and microcrystalline silica [3].

Petrographic methods permit identifying reactive forms of silica, however, it is a difficult task due to the difficulties in identifying particular forms. Thus it is one of the reasons for applying dilamometric methods in order to evaluate the aggregate reactivity and for combining them with petrographic methods. Although this test cannot provide information on the behaviour of aggregate in the presence of alkalis from cement, it permits choosing an appropriate accelerated method, a further method of testing the aggregate reactivity.

RILEM AAR1 [4] recommends microscopic testing of aggregate samples reactivity, basing on: separating particles and observing their cross section (this procedure is relatively uncertain and inappropriate for examining unknown or complex aggregates), spot testing (considered to be the most accurate of the methods) and the whole-rock petrography. On the basis of the tests the aggregates are classified into one of the classes: I - non-reactive aggregates, II - potentially reactive aggregates, III - reactive aggregates.

3.2. The Analysis of Mortar and Concrete Samples in Ultra-Fast Laboratory Tests

a) the accelerated method ASTM C1260 [5]

The method takes advantage of a feature characteristic of most chemical reactions – the reaction goes faster in higher temperature.

Mortar bars with the dimensions 25x25x250mm (in RILEM AAR-2 the dimensions are: 40x40x160mm) [4],with the w/c ratio which equals approximately 0.5, are kept in 1M NaOH solution in the temperature of 80°C. The changes in the samples length are marked on day 4,7,11 and 14. The point of reference is the initial measurement, performed on samples that have been kept for 24 hours in water in the temperature of 80°C. In the ultra-fast method the obtained difference in lengths bigger than 0.1% and smaller than 0.25% indicates a medium reactive aggregate and an expansion of more than 0.25% indicates a very reactive aggregate. At the same time an expansion of less than 0.1% of the initial value indicates a non-reactive aggregate.

In contrast to ASTM C227 – in ASTM C1260 the degree of expansion decreases together with the fall of the w/c ratio (probably the degree of alkali ions migration in the sample goes down). It has also been noted that high temperature and NaOH concentration cause the decrease in calcium ions migration in the material pore solution and consequently the increase in the silica solubility and diffusion from the solution outside. This effect leads to the decrease in the monitored expansion. Thus, the method may produce erroneous results.

According to the conducted tests, ASTM C1260 method gives approximately 36% of falsely negative results[6]. This high number of failures can be reduced by reducing the reactivity criterion from 0.1% to 0.06%. with 14-day tests and 0.13% with tests prolonged to 28 days.

Some researchers of aggregate reactivity have confirmed that it is advisable to prolong the testing time [7].

b) testing mortar samples in a digester

This method is presented as the most promising one for defining the alkali-silica reaction. Mortar samples are prepared according to the ASTM C227 method, however the w/c ratio equals 0.50 and the alkali content equals 3.50% Na_2O_e. The lengths of samples that have been kept in a digester for 5 hours with the pressure of 0,17MPa and the temperature of approximately 1300°C are then measured. It has been concluded that this test is more reliable than the accelerated method ASTM C1260. The suggested expansion time of 0.1% has been recognized as tolerable [8].

c) European standards

According to the European standard AAR-4 (the accelerated method) and AAR-4 Alt. (the alternative method) concrete samples with the dimensions (75±5)x(75±5)x(250±50) mm are stored in the temperature of 60°C. In the AAR-4 method the samples are stored in containers over the water surface, and in the alternative method the samples are wrapped in a wet cotton fabric and polyethylene. The expansion degree is measured during the period of 20 or 15 weeks. AAR-4 test results showing an expansion higher than 0.03% indicate a reactive aggregate [9].

3.3. The Long-Term Analysis

a) ASTM C227 method [10]

ASTM C227 uses cements with the alkali content of 0.60% of Na_2O_e. The standard does not define the value of the w/c ratio in a very precise way. 24 hours after the molding the bars (25x25x250mm) are stored at 38°C±2°C in special containers over the water surface. The lengths measurements of the samples that have been cooled down to the temperature of 23°C are taken after 14 days and then after 1,2,3,4,6,9,and 12 months and if necessary also every six months after that. The difference in lengths between the initial sample and the one measured after 12 months bigger than 0.1% permits the conclusion that the aggregate is reactive. The difference smaller than 0.1% indicates the lack of reactivity.

Tests conducted in many centres allowed the researchers to conclude that this test does not make it possible to predict the reactivity in the case of slowly reacting aggregates such as greywacke or argillite [8]. At the same time, the use of containers that are equipped with felt for the storage of samples is connected with washing alkalis out of mortars, which results in the decrease of the expansion grade. Therefore it is believed that the alkali content should be increased to 1,25% and the w/c ratio should not exceed the value of 0.50.

b) the ASTM C1293 method of testing concrete samples [11]

In this method the concrete samples with the dimensions 75x75x285mm are stored over water, with 100% moisture, in sealed containers and in the temperature of 38°C. The lengths measurements of the samples are taken on day7,28,56 and then after 3,6,9,and 12 months and if necessary also every six months after that time. The expansion which after a year's time equals 0.04% means that the aggregate is potentially reactive. The use of this method is significant when petrographic tests have not provided the rationale for claiming that the aggregate is reactive, whereas mortar tests have given the results confirming the reactivity. The long time of duration is undoubtedly a disadvantage of this method.

Summary

In the presented methods of testing the potential reactivity of siliceous aggregates, the crucial role is played by the determinants of the alkali-silica reaction. Due to the fact that the alkali-silica reaction becomes evident after a longer period of time, increasing parameters such as temperature, pressure, alkali concentration and moisture in the testing methods described in this papers allows the researchers to conclude about the behavior of aggregate in real-life structures.

References

[1] BERNHARDSVEJ, C. *Chemistry of alkali – silica re action and testing of aggregates*. Cement and Concrete Composition, 27(2005) 788-795.

[2] ASTM C295. *Guide for Petrographic Examination of Aggregates for Concrete*. Annual Book of ASTM Standards, vol. 04.02(1998).

[3] LUKSCHOVA, S., PRIKRYL, R.Z. ,PERTOLD. *Petrographic identification of alkali – silica reactive aggregates in concrete from 20th century bridges*. Construction and Building Materials, 23(2009) 734-741.

[4] LINDGARD J., NIXON P.J., BORCHES I. The EU "PARTNER" *Project – European standard tests to prevent alkali reaction in aggregates: Final results and recommendation*. Cement and Concrete Research, 40(2010) 611-635.

[5] ASTM C1260. Standard Test Method for Potential Reactivity of Aggregates (Mortar Bar Method). Annual Book of ASTM Standards, vol. 04.02(1993).

[6] LENKE, L.R., MALVAR, L.J. *Alkali silica reaction criteria for accelerated mortar bar tests based on field performance data.* 2009 Word of Coal Ash Conference – May 4-7(2009) in Lexington, USA .

[7] CHANG-SEON SHON, DAN G. ZOLLINGER, SHONDEEP L. SARKAR. *Evaluation of modified ASTM C 1260 accelerated mortar bat test for alkali-silica reactivity.* Cement and Concrete Research, 32(2002) 1981-1987.

[8] TOUMA, W.E. Alkali-silica reaction in portland cement concrete: testing methods and mitigation alternatives. The University of Texas at Austin, (2000) 16-26.

[9] http://homepage.tudelft.nl/n89v3/Page6.html.

[10] ASTM C227. Standard Test Method for Potential Reactivity of Cement – Aggregate Combinations. Annual Book of ASTM Standards, vol. 04.02(1997).

[11] ASTM C1293. *Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction*. Annual Book of ASTM Standards vol.04.02(2002).

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Sensitivity of Input Date in Decision-Making Program HDM-4

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Abstract. HDM-4 is a computer software for Highway Development and Maintenance Management System. It is a decision making tool for checking the Engineering and Economic viability of the investments in road projects. The World Bank for the global use has developed it.

Keywords: HDM-4, Road Network, IRR, Net Present Value, Roughness- IRI.

1. Introduction

In these days the constantly increasing trend of using the pavement management system leads to the requirement to evaluate the financial return of spent investments into the project realizations, reconstruction or repairing of the road network. In Slovakia this evaluation is performed by using of the Highway Development and Management Model HDM-4 [1] software.

This software is used on the basis of the relationship between costs and expected benefits of the used road, with taking into consideration relevant factors (like economics, environment, etc.) and calibration of predictive models that works with the degradation curves implemented into the software. On the basis of these curves, this software sets the future progress for the several parameters of the road and then evaluates the economic efficiency of the investment [2].

2. HDM-4

The HDM-4 method is generally accepted by the leading grant organizations and governments of many countries and became one of the key tools (basic tools) for decisions on financing of road infrastructure [3]. The software was (had been) developed since 1993 on University of Birmingham.

The World Bank, Asian Development Bank, Swedish National Road Administration, DFID, and many others were also involved in the development of the software and also helped to finance it. The HDM-4 software (Highway Development and Management Tool) is distributed worldwide since February 2000 and his basic form was designed for developing countries, namely Bangladesh, so it counts also with gravel pavements or with non-motorized traffic such as animal sledding, rickshaws and similar. Therefore it was necessary to adapt the software for the specific conditions of a given country; so that the input parameters match, for example, with the specific type of pavement, price values, climatic conditions etc. [4].

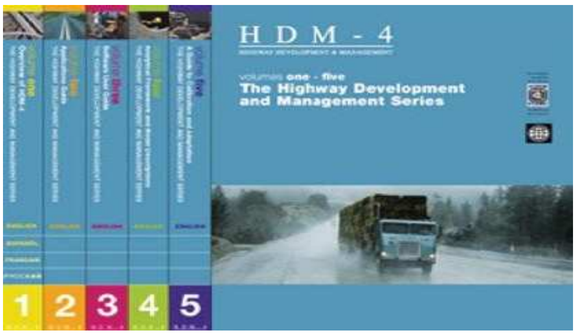

Fig. 1. Development of HDM-4 [6].

3. Effect of Variable Parameters for Economic Evaluation of Road Construction

One of the essential parts of HDM-4 software is the effect of changing of the road surface conditions, defined by menas of variable parameters of the road on the economic rating of the construction. These variable parameters are in HDM-4 software, for example: roughness, total area of cracking, raveling area, potholes, edge break area, rut depth, texture depth, skid resistance and drainage. The user is able to change the various program parameters for the road pavement in defined ranges, which allows performing a preset of the HDM-4 software (Fig. 2).

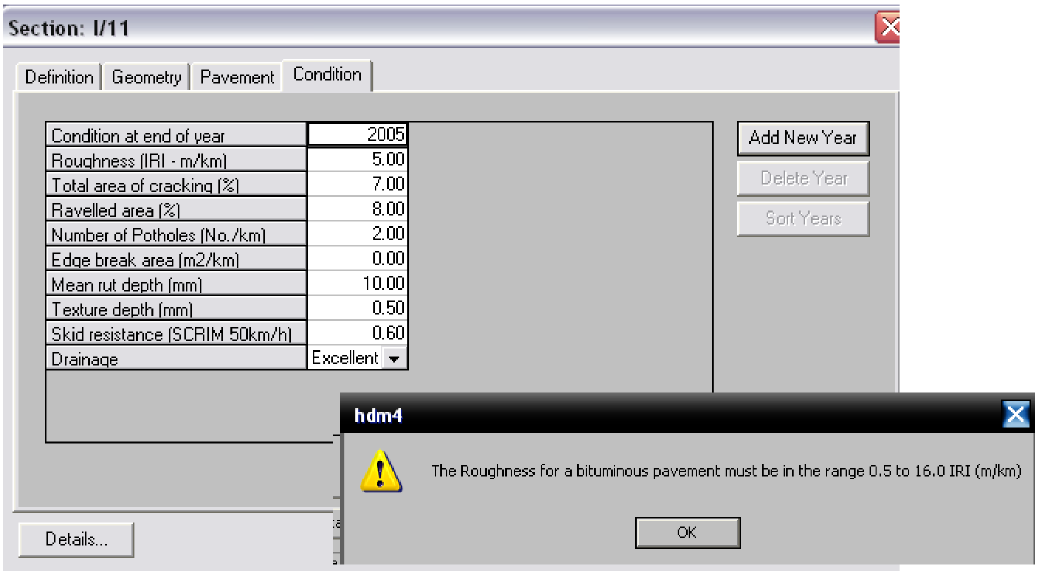

Fig. 2. Pavement condition in HDM-4 [6].

From the total amount of output datas, which the HDM-4 software offers, is the output "Economic Indicators Summary" the most important for economic rating of the monitored construction.

For each variant of solutions for the given section of the construction with using of different repair technologies, there is expressed a number of 9 economic indicators, whereby the Net Present Value "NPV and the Internal Rate of Return" IRR " are the most important for this rating of the construction (Fig. 3).

H D M - 4
HIGHWAY DEVELOPMENT & MANAGEMENT

Economic Indicators Summary

Study Name: Brodno
Run Date: 07-10-2010
Currency: Slovenskß koruna (millions)
Discount Rate: 6,00%.

Alternative	Present Value of Total Agency Costs (RAC)	Present Value of Agency Capital Costs (CAP)	Increase in Agency Costs (C)	Decrease in User Costs (B)	Net Exogenous Benefits (E)	Net Present Value (NPV = B+E-C)	NPV/Cost Ratio (NPV/RAC)	NPV/Cost Ratio (NPV/CAP)	Internal Rate of Return (IRR)
Base alternative	0.000	0.000	0.000	0.000	0.000	0.000	0,000	0,000	0,000
Alternativa 1	3.129	3.129	3.129	1.107	0.000	-2.022	-0,646	-0,646	-77,6 (1)

Figure in brackets is number of IRR solutions in range -90 to +900

Fig. 3. Economic Indicators Summary (Net Present Value,Internal Rate of Return) [6].

Net Present Value reflects here the effectivity of the monitored construction in the form of difference between the updated total cost and the updated total income over the economic lifetime of the investment. Internal Rate of Return takes into consideration the changing value of currency in time and represents the percentage gain of the monitored construction, which can be expected after the payment of all costs. [7]

4. The Effect of The Roughness on the Economic Rating of the Construction

One of the indicators of road safety, passenger comfort and fuel savings is the smoothness of pavement surface. It's an attribute of pavement surface, which affects the safety and comfort of driving through her periodicity and constancy. But at the moment when the defects begin to occur on the pavement surface, defined as differences of the real surface from the theoretical, we are talking about "pavement roughness". This roughness can be then entered into the HDM-4 software in the form of an IRI value (m/km), which represents the value of international longitudinal roughness index of the evaluated section of the construction. As mentioned above (Fig. 2), it's possible to enter this variable parameter into the software as a value between specified ranges of 0.5 to 16 m / km and then observe the impacts of changes from this parameter on one of the economic factors. In this part of the research there was determined the dependence of roughness IRI on the Net Present Value and also on the Internal Rate of Return, whose real process is shown on Fig.4.

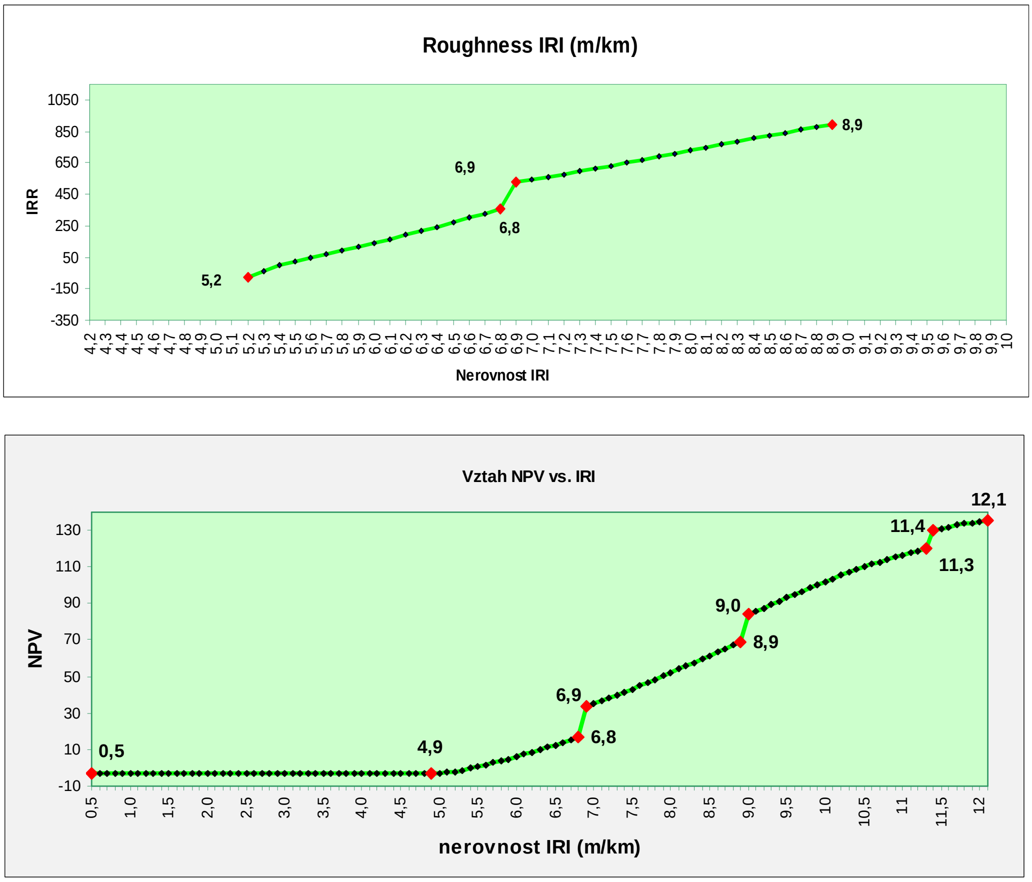

Fig. 4. Effect of roughness changes on the IRR and NPV.

However, these processes are depending on a type of used maintenance standard and criteria that are set in this standard. In this case there was used the ZZ1A maintenance standard - standard for first class roads, sets for Slovak conditions, where the crucial criteria for maintenance interventions into the road is a value of IRI= 7m/km. In practice, this means that at the moment when the value IRI, in any year of the economic evaluation, reaches this limiting value, the repair defined from user (for example: mill and replace) starts and the NPV value increases with a jump.

However, the first part of the graph shows, that the change of roughness values IRI in the range 0.5 to 4.9 affecst in no way the NPV, and therefore has no impact on the economic rating of the construction. In real use of this software this fact would means that the input of the pavement roughness of IRI ≤ 4.9 m / km is irrelevant.

Acknowledgement

HDM-4 software tries to be opened for users from all countries and enables entirely free settings of own parameters, starting with technical parameters of vehicles, economic inputs, but also a predefinition of repair technologies and reconstructions for the road infrastructure. However, these free adjustment has brought also many irregularities which have to be further specified and through sensitivity tests adjusted so that the software provides the real condition of the planned operation. Not only for the roughness, but also for all other variable road parameters, it could be possible by parametric studies to determine the "usable" ranges of individual parameters and so to reduce the large number of input data, which are necessary to be entered into the software. These specifications will significantly help to distribute and to improve the existing methodology.

References

[1] http://www.worldbank.org/transport/roads/rd_tools/hdm4.htm, 18.4.2010.

[2] MORAVČÍK, M., ŠKORVAGOVÁ, A., KLIMENTOVÁ, V.: Porovnání modelu HDM-4 a programu SEH-PS, In: 5.odborná konference doktorského studia s mezinárodní účastí, Brno 2003.

[3] http: //www.cideas.cz/free/okno/technicke_listy/1uvt/1321.pdf.

[4] Dopravní noviny 31/2004, In: Ekonomickou efektivnost staveb nově hodnotí HDM-4.

[5] http: //www.lpcb.org/lpcb-downloads/presentations/99piarc-rue.pdf.

[6] software HDM-4, licence UNIVERSITY OF ZILINA.

[7] MP 1/2009: Metodický pokyn preužívanie programu HDM-4 v podmienkach SR.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

The "Optimism Bias" Phenomena in Feasibility Study Phase of Civil Engineering PPP Project

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Abstract. The article focuses on the phenomena of "Optimism bias" in procurement of civil engineering constructions through public private partnership projects. Firstly, the most sensitive PPP project procurement period the feasibility study is explained. Then I deal with the optimism bias phenomena and I recognize the causes for its presence, what are the impacts and what is the challenge when dealing with it, in process of project procurement. The most significant part presents the results of the research in the field of risk management through two conjunctive methods of reducing the negative effects optimism bias has on the acquisition of civil engineering structures.

Keywords: Optimism bias, Public Private Partnership, Feasibility study, Civil Engineering, Risk Management.

1. Introduction

The success of civil engineering project procurement heavily depends on the consistence of pre-project preparations. The main parts of those preparations apart from the procedures for design and construction are the feasibility study, project cost management and risk assessment.

Its assumed that if this part of project procurement is executed by experienced responsible and motivated experts with enough time and accurate inputs, the procurement will be prepared in a way that exclude any foreseeable problems, errors and inaccuracies which would lead to project failure.

However, throughout the world, in the history of alternative project financing methods, namely PPP (Public Private Partnership), we experienced too often problems ranging from cost or time uplifts to all-out project failures even if the previously mentioned conditions needed for successful project procurement are met.

What is the factor which could have such impact? The answer we seek may be found in the human psychology.

2. Optimism Bias as a Factor in Project Preparation Period

The main part of the project preparation period in the process of PPP project procurement is the realization of feasibility study. It's also the most sensitive phase in which the phenomena of optimism bias can cause most damage.

2.1. Feasibility Study

The feasibility study assesses whether conventional public procurement or a PPP is in the best interests of the institution for the delivery of the service. A feasibility study needs to be authentic and thorough. It is the basis for government's making an important investment decision, not just a bureaucratic requirement. Regardless of the term and scale of a project, there are long-term implications and a great deal at stake when the procurement choice is made.
The feasibility study must demonstrate whether the PPP choice:

- is affordable;

- transfers appropriate technical, operational and financial risk to the private party;
- gives value for money.

The feasibility study is a critical part of the project preparation period of the PPP project cycle:

- It provides information about costs (explicit and hidden), and gives an indication of whether costs can be met from within institutional budgets without disruptions to other activities.
- It allows for the identification, quantification, mitigation and allocation of risks.
- It prompts institutions to consider how the project will be structured.
- It identifies constraints which may cause the project to be halted.
- It ensures that the project is developed around a proper business plan.

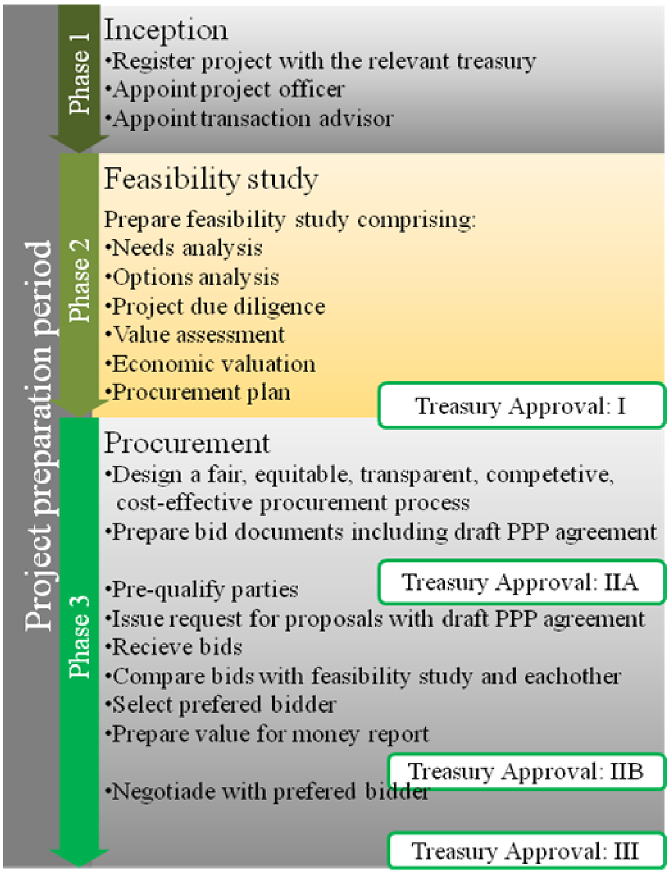

Fig. 1. Comprising of the feasibility study and its position in the preparation period of PPP project.

A feasibility study is an evolving, dynamic process. While it is done primarily to decide whether or not to proceed with a PPP, should the PPP procurement choice be made, it is also used throughout the procurement phase: for continuous risk tracking; to determine value for money; cost-benefit testing platform for changes made during procurement and realization and as a document for negotiation with bidders.

2.2. Optimism Bias

Optimism bias, originally referred to as unrealistic optimism, is the tendency of individuals to underestimate the likelihood of adverse events. It's a term used to describe the demonstrated, systematic tendency for project appraisers to be overly optimistic about project costs, duration and benefits (outputs and receipts/income). As a consequence of this bias, some individuals might disregard precautions that might curb these risks.

Civil engineering projects are inherently risky due to the long planning horizon and complex interfaces. Often the project scope or ambition level will change significantly during project development and implementation. Changes may be due to uncertainty at the early project stages on the level of ambition, the exact corridor, the technical standards, project interfaces and geotechnical conditions, etc. Hence, a certain degree of uncertainty exists which will typically be reduced through the project cycle.

In the field of civil engineering project procurement we can characterize optimism bias as systematic tendency to view things in an overly positive light. It can arise in relation to any aspect of a project but it particularly applies to:

- capital costs ;
- works' duration;
- operating costs;
- under delivery of benefits.

Projects are inherently risky and the risks increase with the size and complexity of the project. Studies have found that the two main causes of optimism bias in PPP project are:

- poor definition of the scope and objectives of projects in the business case, due to poor identification of stakeholder requirements, resulting in the omission of costs during project costing; and
- poor management of projects during implementation, so that schedules are not adhered to and risks are not mitigated.

From other standpoint, the causes can be broadly split into two categories: technical and institutional.

Technical	Institutional
Risks and uncertainties associated with forecasting costs, income etc.	The desire to see projects happen
Changes in project scope	Institutional pressures
Poor project management	The decision-making process, etc

Tab. 1. Technical and institutional causes of optimism bias.

While the technical causes can be deal with proper use of advanced risk and project management techniques, the political-institutional factors have in the past created a climate where only a few actors have a direct interest in avoiding optimism bias. However, it is the norm in projects, rather than the exception, that unplanned events do occur and experienced project managers should consider the effect of these when appraising projects.

The challenge is therefore to:

- make explicit allowance for optimism bias in appraisal in a proportional and cost effective way;
- consider whether a project represents value for money once an allowance for optimism bias is included; and
- be aware of and work to reduce the causes of both technical and institutional optimism bias.

The evidence base of optimism bias consist of post project studies which compare the project outcome with the presumptions, analyses and assessments formulated during the project preparation period. This evidence base works as database for the research focused on mitigate the negative effects of optimism bias. The establishment of probability distributions for cost and time requires access to credible data on cost and time overrun for a sufficient number of projects to draw statistically meaningful conclusions. These projects are represented by all the civil engineering projects realized through PPP financing from early 80's till present days and the database is growing with each started project.

3. Dealing With Optimism Bias

The main aims of the present Guidance Document are to:

- provide empirically based optimism bias up-lifts for selected reference classes of civil engineering projects; and
- provide guidance on using the established optimism bias uplifts to produce more realistic forecasts for the individual project's capital expenditures.

3.1. Estimating Uplifts

The empiric way to deal with optimism bias is to use the cost and time overruns of former projects in a distinct groups where the risk of cost overruns within each of the groups can be treated as statistically similar, on our project.

For each of the groups, a reference class of completed projects has been used to establish probability distributions for cost and time overruns for new projects similar in scope and risks to the projects in the reference class.

Based on this, the necessary uplifts to ensure that the risk of cost overrun is below certain pre-defined levels have been established.

The results are too big for this article, but as an example for a standard not combined civil engineering project is the works duration uplift 20-1% and the capital expenditure uplift 44-3%. The wide spread represents the diverse overruns of various civil engineering projects in history. For a particular project is needed to start with the maximal uplift and possibly reduce it with mitigation factors of 1.0-0.0. The mitigation factors represent favorable odds applying to reduce the uplift up to the lowest possible bound. These odds may be things like good site characteristics, former experience with contractor, low design complexity, use of reliable engineering technology etc.

3.2. Adding the Outside View

As i stated earlier, the political-institutional factors have in the past created a climate where only a few actors have a direct interest in avoiding optimism bias. The decisions based solely of the project team or other experts closely associated with the project (the inside view) is therefore prone to optimism bias. Adding an outside view (reference forecasting) free of political-institutional causes for optimism bias, where information on a class of similar or comparable projects are used to derive information on the extent to which likely - but presently unknown – future events may increase project costs, delay project time schedule or reduce project benefits compared to the base scenario. The outside view does not try to forecast the specific uncertain events that will affect the particular project, but rather tries to place the project in a statistical distribution of outcomes from a group of reference projects. Taking an outside view requires the following steps for the project:

- **Identification of a relevant reference class of past projects**

The key is here that the class is broad enough to be statistically meaningful but narrow enough to be truly comparable with the specific project.

- **Establishing a probability distribution for the selected reference class**

This requires access to credible data on cost increases (or time Schedule delays or benefit shortfalls if these are the key parameter) on a sufficient number of projects within the reference class to make statistically meaningful conclusions (normally at least 10).

- **Placing the specific project at an appropriate point in the reference class distribution**

This step has an element of intuitive assessment and is therefore liable to optimism bias.

References

[1] MFSR *Obsah a požiadavky na štúdiu uskutočniteľnosti a komparátor verejného sektora*,jún 2009.

[2] HM TREASURY. *The green book,* Treasury Guidance, London:TSO.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Formation of Degradative Models of Parameters of Pavement Serviceability

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Abstract. The degradation of roadway is a gradual decline of road condition which is caused by effects of conveying charge in definite climatic conditions and conditions in sub-base of the roadway. This decline of road condition expresses oneself in change of particular paremeters which define collectivly its operative and functional competence. We can express expected change of particular paremeter by degradative model. Formation of degradative models of parameters of operating competence is one of most important part of road diagnostic and the aplications these models helped by more efficient operation of road network.

Keywords: Degaradation, degradative models, congestion, operating competence, roughness, rut depth.

1. Introduction

Nowadays, a qualitative road infrastructure is „driving engine“ for economical, economic and social development of a given area. The constant increase of conveying intensity and so the increase of portion of cartage is the biggest problem of existing roads in term of safety and technic.

All qualitative characteristics of roads are reflected by operative capacity of roadways. This capacity is a complex of roadway features which provide fast, fluent, econimic but mainly safe drive of motor vehicles by it.

And so it is really important to follow the development of particular parameters of operative capacity and consequently by evaluation of these parameters to form degradative models. Thanks to these models we can calculate a concrete parameter of operative capacity depending up the conveying charge or time.

2. Degradative Models

Degradative functions and models of variable parameters are one of the basic part of roadway diagnostics and they are acquired by two basic approaches:

1) On the basis of laboratory tests of materials the roadway consists of and which failures are verified by loading and in conditions which are very near to real
2) On the basis of long-running measurations and observations of the roadway conditon with notation of all known effects, consequently the formation of models on the basis of empirical methods, on the basis of theoretical models [1].

After the statistic elaboration, the complex of measured values of some roadway parameter is used to express a change of one or another measured parameter value depending up the time or the conveying charge. The basic relation of the parameter and the time is in absolute values, for practical use in the system of roadway management, the relation of relative values is more suitable.

As it was mentioned, the forecasting of a roadway condition is able by models of invasion-degradation, when we follow it separately according to particular parameters or collectively for one general parameter.

On the basis of knowldege and results of measuring on different roadways, the first attempts for formation of degradative models have been dated since 1977 (Pavemnet Managment Guide) and in 1983 Molenaar defined one of the first models for persistance of a roadway on the basis of fatigue and failure of materials.

In 1987 The World Bank published the Paterson´s work with models of degradation. They were the dependencies empirically found out in different countries in 1976-1981. Considering to different climatic conditions and also to the types of constructions, these models can not be generally relevant [2].

3. Designation of Degradative Models

Generally in term of methodology, during the prognose of the condition it is needed to recognise a few phases:

1.phase: identification (identifing of parameters values)
2.phase: calculations and evaluation
3.phase: interpretation of results associated with analyse
4.phase: formation of a prognostic model [2].

In the first phase of the prognose it is necessary to gain so called time orders of values of a watched parameter namely with the step of a half year or all year. It is important that the time order has to be built-up from values of parameters measured by the same method (the same measuring equipment, machine or methodics)

During the analyse of value changes in this time order it is important to differentiate:

1. basic tendencies of development (increase, decrease)
2. periodical changes (caused by seasonal effects, external factors)
3. irregular changes (accidental changes) [4].

Nowadays I addict myself to a formation of the degradative model of crosswise irregularities (the depth of a rut). I gained the parameter values from Slovak Road Board by measuring equipment Profilograph (fig.1) from all long-watched road sections, on which The Slovak Road Board carries on regular measures every year. Data are dated since 1998 until the last measuration in 2010, so it is interval of 13 years on 22 roadway sections (11 are non-rigid and 11 are rigid roadways).

The first step is very important and it is a selection of valuational roadways, in term of a surface but also in term of a construction (by material and even by depth of particular layers)

Because of a relatively great deal of data, it is difficult to smooth-out incorrect ones, which arised from different reasons (error during measuring, external conditions) and they can negatively influence consistent calculations and formation of a model. Another relevant fact is that a measuring vehicle is no table to record exactly the same rut every year and so some values can be biased. And due to these facts it is helpfull to have a great deal of data about a given parameter.

By this fact I would like to mention others problems during the formation of degradative models of another parameters of conveying capacity. It is relatively good in crosswise irregularities. But for example by condition of a surface when the measuring vehicle Videocar (fig.1) is needed for measuring and recording, the formation of prognostic model on a communication in service is really difficult. And so it is important to observe concrete disturbances (road holes,cracks,...) and their development during the longest time period (min. 10 years) and it is really difficult to realize.

A similar problem occurs by lengthwise irregularity, when the measuring vehicle Profilograph (fig.1) with 15 laser sensors is used. During the long-term measuring of one section, the interception of all 15 sensors on exactly the same section of a road is impossible.

Fig. 1. The measuring vehicle Profilograph and Videocar in use SSC [3].

By abrasiveness there is a problem with two basic characteristics i.e. the parameter MU, which is a representative value of measuring apparatus Skidometer but there is also a coefficient of lengthwise attrition fp, which characterizes the texture of a road. The evaluation of measurations by finished computing program shows that in complex MU and in detecting dependencies Mu (e.g. on speed) there were no responsible results which should be the base for models of degradation and for functions of degradation [6].

From the mentioned facts the result is that the degradative model of crosswise irregularity is the most objective and the most valuable in term of accuracy and credibility (fig.2).

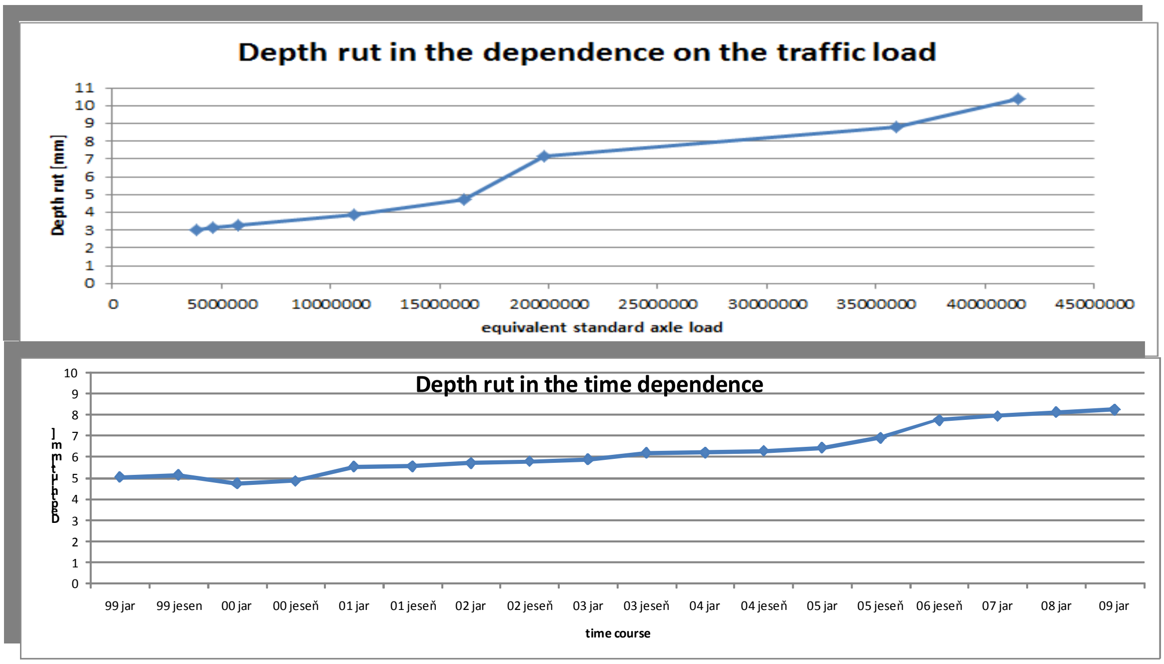

Fig.2. The dependence between transverse inequality and the time dependence and traffic load.

In the figure 2 there are displayed time dependence and the dependance up the conveying charge. In both cases the increasing tendency is observed in enlarging of the depth of ruts. It is observed most in dependence up the cenveying charge when the depth of a rut on a watched section increases rapidly by specific amount of proposed corrections. So the degradative model shows the necessity of roadway reconstruction, reparation or modernisation because in case of passivity and by the constant increase of the number of vehicles, the depth of a rut acquires values bordering on inconvenient condition (in slovak conditions for roads of 1. class it is 20 mm).

4. Experience from Abroad

A research of roadways conditons and of their changes during a long period was the subject-matter of common program of The European Union countries. The first task of this program was to accept the method of measurations and long-term observations of road pavements [5].

This program has began in 1996, in the report from 1997 there is mentioned inter alia the overview of existing degradative models resp. of models for prediction of roadways conditons. Their formation has began in 1980 so it is a long research. The countries as Austria, Great Britain, Finland, Denmark, Belgium are connected in this program.

A lot of mathematical functions was used for representation of changes of the particular parameters or the complexes of parameters and in case of empirical relations there is a number of parameters which are relevant for assertive conditons [2].

Acknowledgement

The degradative functions and models of parameters of conveying capacity are undetachable part of development analyse of the road condition during its lifetime and one of the most important coefficient for decision about its reconstructions, repairs or modernizations.

The ambition for formation of degradative models is to form such functions which have generally relevant form and we can use them in larger spectrum of roadway classes on various parameters of the roadway. But it is really difficult because it is impossible to keep the same conditions (of measuring or of external effect) during long-term measurations. And because of this, regular measurations, observation of the development, evaluation of particular parameters and their archiving is Alpha and Omega for a formation of qualitative degradative model.

References

[1] Čelko J., a kolektív : Povrchové *vlastnosti vozoviek (Prevádzková spôsobilosť vozoviek)*, ŽU v Žiline, 2000.

[2] Gschwendt I., a kol: *Diagnostika vozoviek – degradačné modely*, správa úlohy, Bratislava, 1997.

[3] http://www.cdb.sk/sk/CDB-Cestna-databanka/Zber-udajov/.

[4] Schlosser F., a kolektív : *Technológia stavebných prác*, ŽU v Žiline, 2005.

[5] [5] DECKÝ, M., GAVULOVÁ, A., PUTIRKA, D., PITOŇÁK, M., VANGEL, J., ZGÚTOVÁ, K.: *Navrhovanie a rozpočtovanie asfaltových vozoviek*. Vysokoškolské skriptá, Stavebná fakulta ŽU, 2010, s. 300, ISBN 978-80-970388-0-9.

[6] http://www.ssc.sk/files/documents/technicke-predpisy/rozborove_ulohy/sprava_rvt_2009_modely_degradacie.pdf.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

The Selected Problems of Cement Industry in Poland Resulting from the Usage of Substitute Fuels from Waste in Clinkier Burning Process

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Abstract. The selected problems with the substitute fuels usage from waste in cement industry in Poland are introduced in this article. In addition, this paper states the proposition of implementing to the Polish law the requirements included in the technical specifications prepared by CEN/TC 343 (European Committee for Standardization) within the European Standards activities. The paper is based on the one of the chapters of my doctoral thesis: "The system of obtaining solid substitute fuels from biodegradable waste to the thermal processing in cement kilns".

Keywords: Alternative fuels, substitute fuels, cement plant, waste, co-incineration process.

1. Introduction

Since Poland joined European Union, the systematic increase of interest in management of municipal waste has been observed. The connected actions, which aim to reduce the amount of generated waste, including the search of the possibility of energy recovery, are taken by local government, research institutions as well as production plants. Cement industry is specifically and properly adjusted to utilize waste as substitute fuel. In addition, it is presently expected that the cement industry provides support in solving the waste management problematic issue in Poland. The knowledge of the co-incineration of the selected flammable fractions of waste used in clinkier burning process is commonly known, confirmed by the research results in the wide range of publications, thus, it is not the subject of this paper. The paper presents the selected problems of cement industry in Poland resulting from the usage of substitute fuels on the base of the preliminarily processed waste.

2. Cement Plants Problems as a Result of Usage of Substitute Fuels in Clinkier Burning Process

The cement industry in Poland presently includes 11 production plants which, for many years, have been successfully prospering among the international capital groups. Between 1999-2000, the majority of these plants made the investments, amounting to 858 milion PLN, which have also been directed to environment protection. As a result, the cement plants are technologically well prepared to lead the co-incineration process of the selected flammable fractions of waste [1]. The waste which can be utilized in thermal recovery process inside the kilns is, generally defining, the waste which can not be used as a recyclable material and which contains energy.

The flammable materials, named alternative fuels (the waste code-191210), which are currently delivered to the cement plants in Poland, can be divided into 5 groups:

group 1: wood, paper, cardboard and cardboard boxes,
group 2: textiles
group 3: plastics and rubber
group 4: other materials (for instance: paints residues, used absorbers, used solvents, used oils)
group 5: high calorific value fractions from non hazardous mixed waste [2].

The processing of these kinds of waste to the usage as fuels is usually made outside the cement plant. It is realised either by the fuel suppliers or the other specialized companies responsible for deliveries of alternative fuels to the cement plants. The main features of the substitute fuels controlled during the initial phase of its formation are: physical state, calorific value, moisture, amount and chemical content of ashes, homogenity, ability to be processed and transported, granulation and density. As for the quality issue of the delivered fuels prepared on base of the selected waste, it should be underlined that not fulfilling the basic requirements necessary to the co-incineration of these fractions and recommended by EURITS (European Union for Responsible Incineration and Treatment of Special Waste [3]) can either badly influence on or even disturb efficiency and effective performance of the kiln, namely:

- the increased chlorine, sodium and potassium content can lead to the creation of so called coatings inside the kiln, consequently disturbing the performance of all the instalation. The excess of the permitted amount of chlorine and alkalia in alternative fuels can lead to the creation of by-pass dust, which must be strictly and immediately removed from the installation.
- the high water content in substitute fuels has considerable influence on decrease of the production and efficiency indicators of the kiln.
- the low calorific value of alternative fuels (according to EURITS lower than 15MJ/kg) can lead to disturbance in providing the optimum conditions of clinkier burning process. The co-incineration of lower calorific value fuel prepared on base of selected waste is possible, however, in this case, the utilization costs are significantly higher than the achieved ecological effect.
- The excessive ashes content in substitute fuels can cause the worsening of the chemical composition and consequently the quality of the final product of clinkier grinding process - cement [4].

It should be clearly stated that the cement plants, on a large scale, receive varied, flammable fractions of the substitute fuels derived from the alternative fuel installations, including the municipal waste sorting plants. As a result, the cement plants take responsibility for the management of all these substitute fuels, and, in accordance with the paragraph 45 of the amended Waste Management act, they do have „*the duty to undertake the essential precautions aiming to prevent or limit negative influence on environment, particularly referring to air pollution soil, surface and ground water contamination, odours, noise, and also the direct hazard to human health...*" Therefore, referring directly to air protection against pollution coming from alternative fuel co-incineration processes, it ought to be emphasized that, currently, the cement plants in Poland, thanks to the necessary investments (in 1999-2000) in precipitators for that matter, fully realise the tasks included in the strict requirements. They are written in:Ministry of Environment Regulation from 4th August 2003 on the emission standards from installations (the Official Journal no. 283, item 2842), Ministry of Environment Regulation from 23rd December 2004 on the requirements for conducting measurements of emission amounts (the Official Journal no. 163, item 1584), Ministry of Economy, Labour and Social Policy Regulation from 22nd December 2003 changing the regulation on the requirements for conducting the process of thermal waste treatment (the Official Journal no 1, item 2). The main and crucial difficulties, which must be dealt with by majority of the cement plants in Poland, (referring to meeting the requirements included in the paragraph 45 from the above Waste Management act), undoubtedly comprise:

- the lack of the quality management system concerning the delivered fractions of substitute fuels, the system adapted and prepared to the thorough and precise analysis and indentification of the chemical substances which can be potentially hazardous for workers' health, including the biological agents, present, for instance, in meat-bone meal [5].
- the lack of the uniform system concerning the health protection of the staff who works in contact with both alternative fuels and other waste, from the hazardous waste group,

delivered to the cement plants, in accordance with the integrated permission issued by the provincial governor.

- the lack of the protection system for employees against the odours emitted from halls and hoppers where the delivered alternative fuels are being stored.
- the lack of the emergency procedures aiming to limit the hazardous effects of unplanned spillage and leakage from installations to the soil and water environment.

It is undeniably stated that the alternative fuel market in Poland has rapidly been expanding and growing for the recent years, which has been shown by the example of the data concerning the number of the generated substitute fuels on base of waste- 66 040 Mg in 2006 and 99 536 Mg in 2007 [6]. So, it should be crucial to avoid and exclude negative influence of substitute fuels on the kiln performance, surrounding natural environment and particularly staff health. Therefore, it seems to be highly recommended, on the level of: the state legislation, cement plants and alternative fuel suppliers, to put the considerable emphasis on implementation and enforcing of all the requirements by CEN/TC 343 committee, particularly in those which are included in the technical specifications enlisted in the following Tab.1: [7]

	Number of specification	Title of the specification
1	CEN/TR 15441:2006	Solid recovered fuels – Guidelines on occupational health aspects
2	CEN/TR 15508:2006	Key properties on solid recovered fuels to be used for establishing a classification system
3	CEN/TS 15357:2006	Solid recovered fuels – Terminology, definitions and descriptions
4	CEN/TS 15358:2006	Solid recovered fuels – Quality management systems - Particular requirements for their application to the production of solid recovered fuels
5	CEN/TS 15359:2006	Solid recovered fuels – Specifications and classes
6	CEN/TS 15400:2006	Solid recovered fuels – Methods for the determination of calorific value
7	CEN/TS 15402:2006	Solid recovered fuels – Methods for the determination of the content of volatile master
8	CEN/TS 15408:2006	Solid recovered fuels – Methods for the determination of sulphur (S), chlorine (Cl), fluorine (F) and bromine (Br) content
9	CEN/TS 15410:2006	Solid recovered fuels – Method for the determination of the content of major elements (Al, Ca, Fe, K, Mg, Na, P, Si, Ti)
10	CEN/TS 15411:2006	Solid recovered fuels – Methods for the determination of the content of trace elements (As, Ba, Be, Cd, Co, Cr, Cu, Hg, Mo, Mn, Ni, Pb, Sb, Se, Tl, V and Zn)

Tab. 1. The selected technical specifications prepared within CEN TC/343.

3. Conclusion

It can be concluded that the growing trend, both for generating and utilizing of alternative fuels, which has been observed for the recent years, has contributed to the steady presence of substitute fuels in the cement industry. Currently, using the alternative fuels for the sake of cement plants has become obvious, or even necessary, and the rising demand for the substitute sources of energy is leading to the their shortage on the Polish market. On the one hand, we observe that those cement plants which are technologically and economically prepared, are definitely win the market „struggle" to retain the substitute fuels. On the other hand, the increased demand for the flammable material prepared on base of selected waste is causing that the municipal waste sorting plants equipped with alternative fuel installation do not pay sufficient attention to the proper selection and processing of the waste. Consequently, the quality of these substitute fuel is becoming worse, and, furthermore, their chemical composition (especially heavy metal content) does not meet the basic requirements stated in EURITS. Presently, majority of the cement plants, located in Poland are facing and trying to cope with the mentioned issue. Therefore, the solution to the above problems is supposed to be the implementation of (to the Polish state legislation level) the requirements included in technical specifications announced by CEN/TC 343.

References

[1] http://www.polskicement.pl/?s=1/3/6/4/0.

[2] KRUCZEK, H. *Standaryzacja paliw alternatywnych do spalania i współspalania w kotłach energetycznych – potencjał naukowo – badawczy i wdrożeniowy*", Wrocław, 2007.

[3] http://www.incineration.info/publications.

[4] STOWARZYSZENIE PRODUCENTÓW CEMENTU. *Współspalanie paliw alternatywnych w przemyśle cementowym – zrównoważony rozwój*. Polski Cement. Naturalnie dla Środowiska.

[5] ZIELIŃSKA-JANKIEWICZ, K., KOZAJDA, A., *Narażenie na czynniki biologiczne przy pracy w kontakcie z mączką mięsno-kostną*, Zakład Środowiskowych Zagrożeń Zdrowia, Instytut Medycyny Pracy im. Prof. J. Nofera w Łodzi, 2008.

[6] Dane otrzymane z Departamentu Gospodarki Odpadami, Ministerstwo Środowiska, 2009.

[7] http://erfo.info/fileadmin/user_upload/erfo/documents/standardisation/N_230_CEN_TC_343_Published_documents.pdf.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Numerical Simulation of Freezing Process in Early-Age Concrete with Phase Change Phenomena

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Abstract. This paper shows experimental and numerical study of thermal distribution during freezing process in early-age concrete including phase change phenomena. Cylindrical shape concrete samples were instrumented with thermocouples and strain sensor. Predicted temperature profiles were developed with ADINA-T – finite element program. Numerical and experimental results are in good agreement. The influence of differential temperature profiles to strains was preliminary discussed.

Keywords: Freezing, early-age concrete, thermal strains, phase change, numerical simulation.

1. Introduction

During late autumn and early spring, daily temperatures may fluctuate between positive and negative values. These variations are the cause of numbers of problems at the construction sites, especially with concrete works. The curing temperature can differ significantly from the temperature during placing of concrete. Low initial temperature and short time period between daily extremely temperatures inhibit the resistance development which conduce development of microcracks damages during freezing.

There is no actually national or European code or guideline in which that technological problem concerning the influence of negative temperature values in the initial curing of concrete is solved. However, the authors found some information about concrete works during the low temperatures in [5] and [6].

The preliminary experimental and numerical study presented in this paper show the nature of freezing in early-age concrete including comparison between recorded and predicted temperature profiles, correlation between strain and temperature variation in the sample cross-section during the phase change phenomena, as well as discussion about different strain values between series.

2. Experimental Procedure

2.1. Experiment Assumption

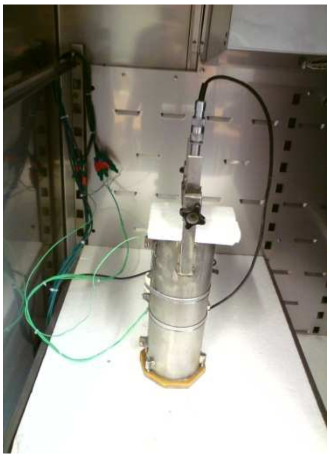
To record the nature of freezing in concrete, two series of five samples was tested. For this paper only two samples observations were used without thawing process records. The concrete composition is given in Tab. 1.

The samples were made in the cylindrical shape metal form with internal radius of 0,05 m and 0,3 m height. The form was insulated from the bottom and from the top with 2 cm polystyrene plates. All samples were instrumented identically. Instrumentation of each sample consisted of 5 K-type thermocouples and 1 mechanical strain sensor in Fig. 1 configuration.

Component	Value	
	Sample 1	Sample 2
w/c ratio	0,60	0,55
Cement, kg/m^3	308	336
Water, kg/m^3	185	185
Sand, kg/m^3	684	676
Aggregates 4-16 mm, kg/m^3	1390	1372

Tab. 1. Composition of the concrete mixes.

All instruments were connected to the Ecograph T recorder then to the computer. Recorded strain values were calibrated from voltage signal to millimeters.



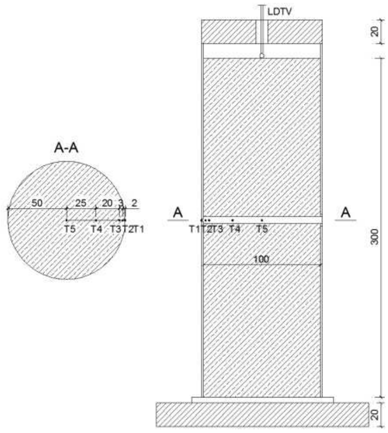


Fig. 1. Test stand and experimental instrumentation schema (T1,T2,..,T5 – thermocouples, LDTV – strain sensor).

2.2. Recorded Strain and Temperatures Profiles

For this subsection only first sample observations were used. Temperature and strain measurement was carried out with sampling frequency of 360 samples per hour. Fig. 2 show that there are four time-temperature regions that are irrespective of the location of thermocouples as well as correlation between strain profile and two of the temperature profiles – surface and center profile.

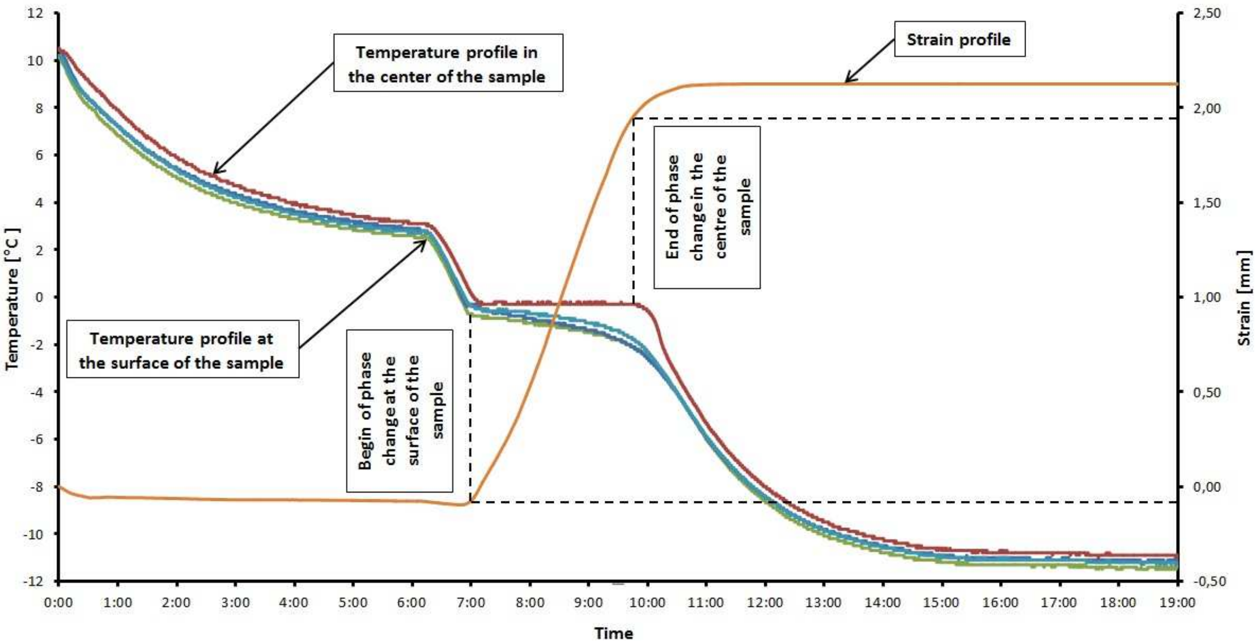


Fig. 2. Measured temperature and strain values.

2.3. Different Strain Values between Series

Measured strain increases during phase change are different between sample 1 and sample 2. First sample strain increase is almost twice as large as second sample strain increase. The reason is in the different compositions of both samples as well as in the different initial and curing temperatures.

Lower w/c ratio and higher initial and curing temperatures for second sample conduce development of resistance. The different profiles of strains increase are shown in Fig. 3.

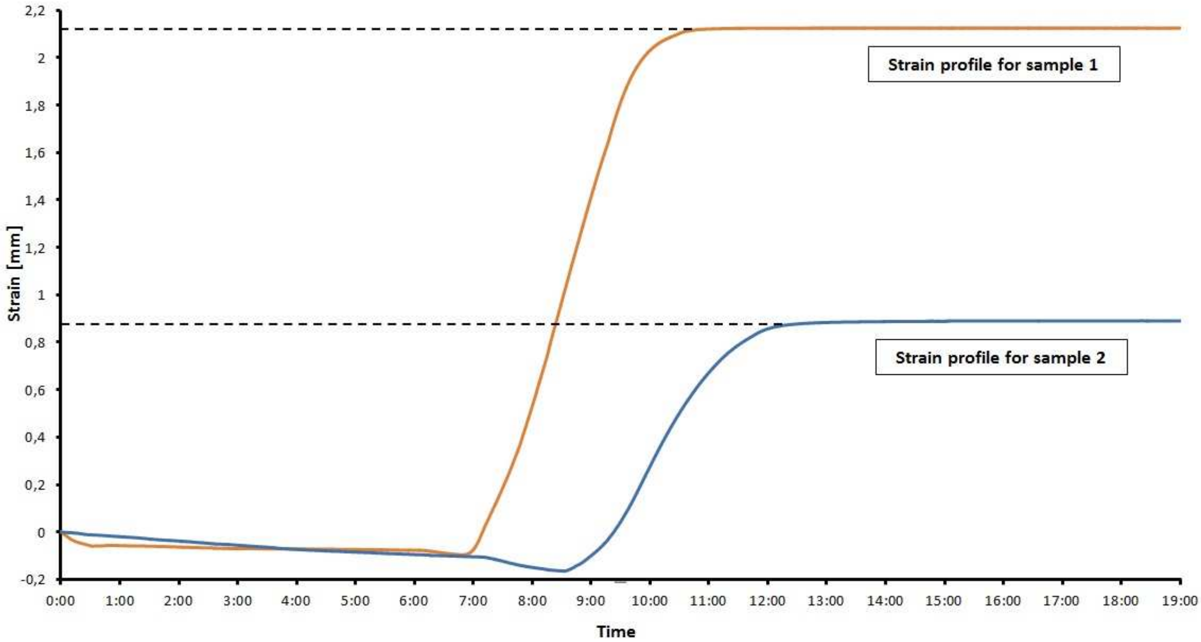


Fig. 3. Measured strain values of both samples.

3. Numerical Simulation

3.1. Properties and Parameters Used in Analysis

For this subsection only first sample observations were used. Thermal analysis was made with ADINA-T program. The properties used in the analysis are given in Tab. 2. The 2D axisymmetric geometry were used as it is shown in Fig. 4. [1], [2].

Notification	Property	Value	Reference
ρ	Weight/volume, kg/m^3	2560.0	-
ρc	Specific heat/volume, kJ/m^3·°K	3500.0	Aleksandrovski [3], [4]
k	Thermal conductivity, kJ/hr·m·°K	9.5	Andreasik [3], [4]
L	Latent heat of fusion for water, MJ/m^3	319.8	-

Tab. 2. Thermophysical properties and analysis parameters used in FEA.

The two-dimensional discretization using 1300 8-node elements were developed. The load that was applied into the model was the temperature profile that was recorded at the surface of the sample.

3.2. Comparison between FEM and Experimental Results

The FEM and experimental results agreement are good and show in Fig. 4. The difference between measured and predicted values is not larger than 0.6 °C, which is acceptable for further analyses.

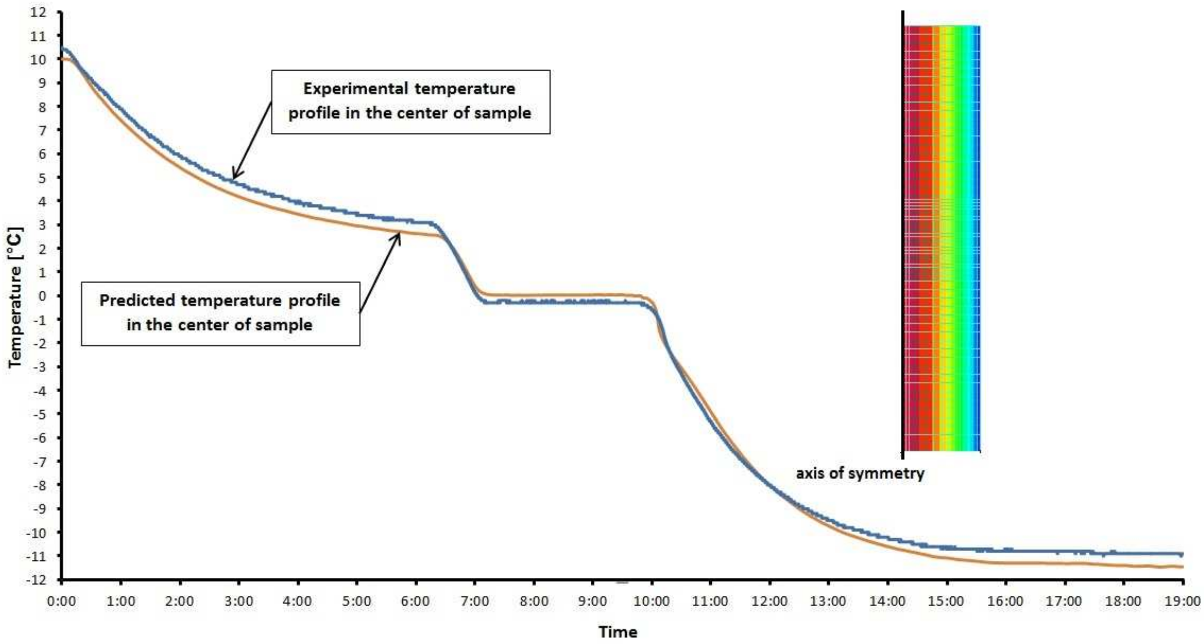


Fig. 4. 2D axisymmetric geometry and comparison between FEM and experimental result .

4. Conclusion

The preliminary experimental and numerical study presented in this paper show that knowledge of the temperature profile at the surface is sufficient to predict temperature distribution in concrete sample during freezing process including change of phase. Future research should focus on creating a numerical model for predicting strains development during freezing and thawing process in early-age concrete and on founding correlation between the magnitude of these strains and 28 day-strength decrease, as well as correlation between temperature distribution and range of damage.

Acknowledgement

The authors wish to acknowledge that their scientific research has been partially financed and carried out as a part of the Project „Innovative recourses and effective methods of safety improvement and durability of buildings and transport infrastructure in the sustainable development" financed by the European Union from the European Fund of Regional Development based on the Operational Program of the Innovative Economy POIG 01.01.02-10-106/09-02.

Furthermore, the authors wish to acknowledge outstanding merits support from their tutor Dr habil. Jerzy Wawrzeńczyk, Eng., the University Prof.

References

[1] ROLPH, W. D., BATHE, K. J. *An efficient algorithm for analysis of nonlinear heat transfer with phase change.* Int. J. Numer. Meth. Engng., 18, 119-134, 1982.

[2] ADINA user interface command reference manual, Vol. II, ADINA-T model definition, report ARD 01-3, June 2001.

[3] KLEMCZAK, B. *Modelowanie efektów termiczno-wilgotnościowych i mechanicznych w betonowych konstrukcjach masywnych*. Wydawnictwo Politechniki Śląskiej, Gliwice 2008.

[4] KIERNOŻYCKI, W. *Betonowe Konstrukcje Masywne – Teoria, Wymiarowanie, Realizacja.* Polski Cement, Kraków 2003.

[5] KUCZYŃSKI, W. i inni, *Budownictwo Betonowe, Tom 1 – Technologia Betonu cz. 1*. Arkady, Warszawa 1972

[6] URBAN, L. i inni, *Wytyczne wykonywania robót budowlano-montażowych w okresie obniżonych temperatur.* MGPiB ITB, Warszawa 1988.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Crossing's Overload in Cities Based on Example Crossing: Sandomierska – Źródłowa Streets in Kielce

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Abstract. Crossings with central islands usually consist a good solution for better functionality in the cities. Problems begin at the moment of overload caused by heavy urban traffic as well as through traffic. Moreover, the traffic is much heavier from year to year. In the paper there are presented results of traffic analysis of functioning one of the most important crossing in Kielce. There are also presented the results of functioning this crossing after amending to traffic regulation, geometry and program group diagram. This elaboration signalizes that the only alternative to improve traffic condition on the crossing could be an elimination some groups of users, building and ensure an access to the beltway or general rebuilding of the crossing to the other type. The elimination of through traffic from the city should automatically reduce the traffic loading on the crossing and reinstate an acceptable traffic conditions.

Keywords: Crossing, traffic conditions, through traffic.

1. Introduction

Crossing IX Wieków Kielc – Źródłowa – Sandomierska – al. Solidarności streets is very important for streetscape in the city of Kielce. It leads urban traffic and through traffic, including generally heavy vehicle traffic. The crossing is situated in the eastern part of the city on the intersection of two main roads: number 73 Wiśniówka – Kielce – Jasło, number 74 Sulejów – Kielce – Zosin – the Polish borderline and the district road number 762 Kielce – Chęciny – Małogoszcz.

A big impediment in traffic, especially during peak traffic, nuisance caused by significant heavy vehicle rate in a traffic and analysis of traffic incidents contributed to the interest in this problem and to the traffic conditions analysis and assessment.

2. Characteristic of the Crossing

It is a four inlet crossing with central island and widened inlets. Traffic is led around a central island 30 metres in diameter. It is a crossing of divided highway and fixe time method of control. The traffic control is coordinated with signal group diagrams of next crossings. The cycle lasts 78,0 sec.

Crossing under discussion was designed at the time when the traffic intensity was much smaller than today and it was built according to the outdated specification. Either the geometry solution or traffic organization of the crossing is not customized to today's regulations.

During stocktaking of crossing on the 2nd May 2008 there was noticed that some issues didn't meet requirements. It is necessary to mention about:

- lack of individual left – turn lanes on two inlets of the crossing,
- length of crossovers on inlets and outlets on the crossing exceeded design limits,
- incorrect length of turning lanes on the inlets,

− wrong location of signal aspect on the crossing.

3. Traffic Conditions Assessment for Existing State of Crossing

On the basis of traffic count provided on the 9th April 2008 and traffic analysis, the traffic conditions assessment was done. Calculations take into account existing geometry, traffic regulation and present signal group diagram. They were done by MOP SZS 04 method. However the analytic procedure omits traffic regulation in operation.

Lack of reserve of capacity for three inlets is worrisome. On the first inlet it is 126 v/h and on others respectively: -173, -226 and -392 v/h. Load factor of crossing exceeds allowed values and it is X_{cr}=1,11. Calculated value of mean waste of time on the crossing is near to the limit of acceptability and it is d_{cr}=72,99 s/v and for the critical group of traffic lane the value is much higher d_{gr}=235sek.

This adverse traffic conditions could be the result of significant part of heavy duty traffic on the crossing, which increased in the period of 2000-2008.

Comparison of heavy vehicle traffic participation on the crossing in 2000, 2006 and 2008 presents tab. 1.

Inlet	**Peak**	**Heavy duty traffic**					
		2000		**2006**		**2008**	
		Number [v/h]	Part [%]	Number [v/h]	Part [%]	Number [v/h]	Part [%]
1. ul. IX Wieków Kielc	morning	51	5,9	39	5,0	66	7,5
	afternoon	58	3,8	48	4,1	64	4,7
2. ul. Źródłowa	morning	86	6,7	77	4,3	92	5,4
	afternoon	93	5,7	84	5,4	111	7,3
3. ul. Sadomierska	morning	58	6,0	82	7,3	98	8,3
	afternoon	62	5,3	85	8,0	92	8,0
4. al. Solidarności	morning	116	10,2	83	6,7	114	8,9
	afternoon	105	8,4	39	6,0	88	5,9

Tab. 1. Participation of heavy duty traffic on the crossing, years 2000, 2006 and 2008.

Comparing year 2000 with 2008 and considering the increase of heavy duty vehicles number and their share increment in the traffic it is necessary to notice important differences, especially on the third inlet. The inlet leads traffic of a main road number 74. During the morning peak scale ~~of~~ heavy traffic participation increased there even to 70%. Situation did not deteriorate only on the fourth inlet.

Results of analyses show that for the group of traffic lanes on this inlet the load factor is the highest. Slight increase of the participation of heavy vehicles on this inlet could contribute to serious disturbances of the traffic on this intersection. Increase of heavy traffic in the per cent comparing years 2000 and 2008 is as follows:

Year	**Inlet nr 1**		**Inlet nr 2**		**Inlet nr 3**		**Inlet nr 4**		**ΣQc**	
	The number of heavy vehicles [v/h] on peaks									
	AM	**PM**	**AM**	**PM**	**AM**	**PM**	**AM**	**PM**	**AM**	**PM**
2000	51	58	86	93	58	62	116	105	311	318
2008	66	64	92	111	98	92	114	88	370	355
Δc [%]	29,4	10,3	7,0	19,3	69,0	48,4	-1,7	-19,3	19,0	11,6

Tab. 2. Increase of heavy duty traffic on the crossing in 2000 and 2008.

The presence of heavy duty traffic in the traffic flow has direct influence on the capacity and on the load factor of the crossing.

Reduced share of heavy duty traffic causes that the load factor on the crossing is lower and it also amends the capacity.

4. Conceptions of Solutions

Considering very high and still increasing traffic intensities (more than 5500v/h for all inlets), as well as the fact that the intersection solutions don't follow up to date design requirements, there are presented three conception of changes within the scope of the traffic organization and the geometrical solution with adapting of the program group diagram, aiming to the optimal use of the road network.

Three attempts of getting the improvement in measures of functioning considered junction were provided. The following options were suggested:

Scope of changes	**Option**		
	1	***2***	***3***
Geometry	without changes	- building the directional island triangular, - allocating left -turning lane on the inlet of the IX Wieków Kielc street and the Solidarności Ave.	- building the traffic island to channelize the traffic , - correction the radius of right arcs - moving closer zebra crossings to the intersection - correction of the width of inlets and outlets, - changing the shape of the centre island, - adapting lengths of the accumulation to current intensities.
Traffic regulation	- allocation the left-turn lanes on two inlets: IX Wieków and Sandomierska streets	- allocation the left-turn lanes on two inlets: IX Wieków and Sandomierska streets - allocation right-turn lanes on Solidarności Ave.	- left-turn lanes and right-turn lanes allocated on each inlet.
Signal group diagram	Correction of the program group diagram length : Option 1a – Tc_{max} – 90s Option 1b – Tc_{min} – 84s	Correction of the program group diagram length: Option 2a – Tc_{max} – 90s Option 2b – Tc_{min} – 78s	Correction of the program group diagram length: Option 3a – Tc_{max} – 90s Option 3b – Tc_{min} – 78s

Tab. 3. Proposed changes on the intersection description.

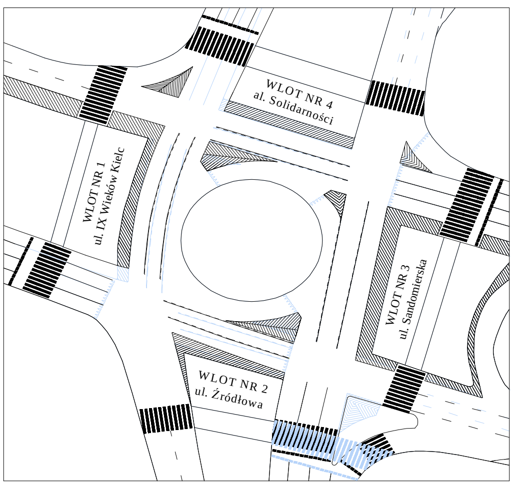

Fig. 2. Scope of changes in geometrical crossing solution option 2.

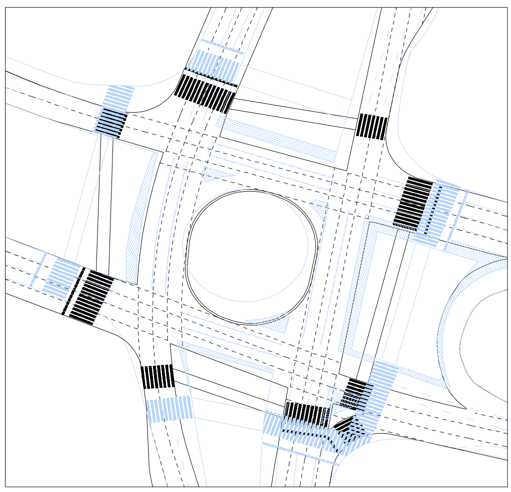

Fig. 3. Scope of changes in geometrical crossing solution option 3.

Warning: The existing state of crossing is marked by the bright colour, proposed changes are black.

5. Proposed Options Traffic Condition Assessment

Results achieved for suggested conceptions as well as the comparison to existing intersection are presented on below graphs. For option 1, where the traffic organization was changed, there were achieved the best results for the cycling about the length of 90sek. However they don't differ much with measures of the assessment of the traffic conditions for the functioning crossing. Both in terms of comparing average wastes of time, as well as the load factor for the intersection, variant 3 turned out to be the intermediate solution among remaining conceptions. The most favourable results were achieved for option 2 enforcing slight amendments in the geometry of the intersection. Maximum extend to the cycling lowered the load factor of crossing to the value below 0.95. However this value isn't satisfactory.

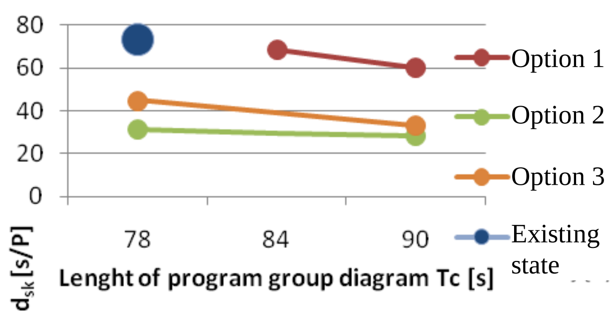

Fig.4. Relation between average wastes of time on the intersection, and the length of the cycling.

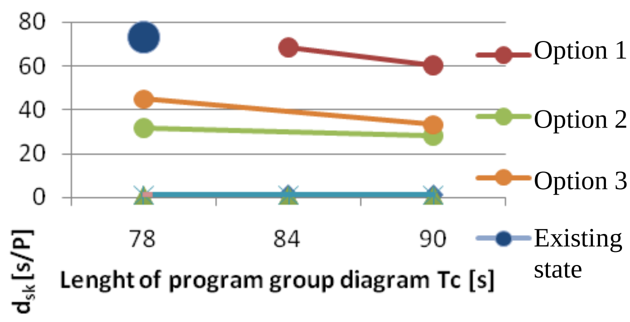

Fig.5. Relation between the load factor of the intersection and the length of the cycling.

6. Conclusion

Higher and higher traffic intensities recorded, arduousness of the heavy duty traffic on arterial (through traffic) for residents of cities, the road noise, and results of conducted analyses shown in measures of the function effectiveness of the intersection, explicitly are inducing for taking radical changes. The most effective solution guaranteeing the improvement in traffic conditions by eliminating certain user groups and reducing the traffic intensity, seems to be to direct through traffic, mainly heavy traffic beyond the urbanised area.

Despite taken attempts to implement amendments to the traffic regulation, geometry and the program group diagram, satisfactory results hadn't been achieved. During the most burdened periods of the day, average wastes of time on the intersection, the load factor and other measures giving the evaluation of the functioning of the intersection are not satisfactory. Analyzed conceptions haven't improved results of analyses in a significant way. It was also noticed, that the junction is functioning better at the maximum length of the cycling for all of shown variants.

Considering the scale of present traffic intensities and their increment possibility in following years, one should expect, that crossing with central islands, won't be able to function efficiently. Till the end of 2011 an investment will be finalized in Kielce, which could make the crossing load much lesser. Future traffic count will show if taken action have a positive effect. Detailed analysis of the effects of increasing participation of the heavy duty traffic on main roads in city's should be done before making the decision about not urban traffic implementation on the street in cities.

References

[1] GACA S., SUCHORZEWSKI W., TRACZ M., *Inżynieria ruchu drogowego*, Warszawa 2008.

[2] STĘPIEŃ M., *Praca dyplomowa magisterska :Ocena warunków ruchu na skrzyżowaniu Sandomierska – Źródłowa wraz z koncepcją zmian w organizacji ruchu. Written under the academic supervision of PhD. Eng H. Major*. Kielce University of Technology, 2008.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Effect of the Stress State Triaxiality on the Value of Limit Strain of Micro-Void Development in S235JR Steel – Numerical Analysis

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Abstract. This paper deals with the analysis of plastic deformation and damage development in S235JR steel in complex stress states. The material Gurson Tvergaard Needleman (GTN) model was applied. The analysis was performed for notched bars subjected to tension. An attempt was made to investigate the impact of stress triaxiality on the value of critical strain of voids growth in the GTN model.

Keywords: Ductile fracture, porous metal plasticity, GTN model, numerical analysis.

1. Introduction

Computing load carrying capacity of steel components of civil engineering structures at the design stage is a well-defined, relatively well-known process. It is far more difficult to make an analysis of existing structures operating under structural failure conditions (overload, damages to load bearing elements). In such a case, it is required to account for plastic reserve of the material load bearing and to apply the principles of fracture mechanics.

It is essential to employ a model of the material plastic flow. In the literature on the subject, one can find many solutions [1, 2], yet their practical applications, especially to complex stress states, could be sometimes restricted.

Models of porous bodies offer a lot more possibilities [3, 4]. They describe fracture process by means of growth and coalescence of voids that are initiated on the boundary of the metallic matrix and the second phase particles (mainly sulphur and manganese compounds) (Fig. 1).

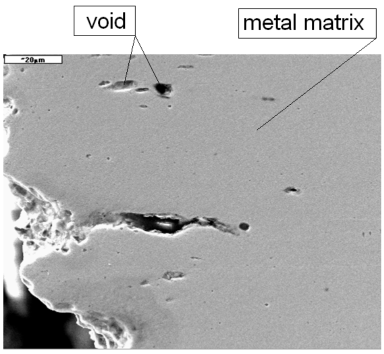


Fig. 1. Microstructure of investigated steel.

One of the solutions in the field is Gurson-Tvergaard-Needleman model of the material. In order to obtain most reliable results, it is of key importance that the values of strains accompanying micro-void development are determined. According to many authors, the process of micro-void growth depends primarily on the degree of stress state triaxiality. That implies a similar pattern of changes will be demonstrated by strain values at the instant of rapid micro-void development and failure. The present paper aims at determining values of effective strains that accompany micro-void growth and the component failure as a function of a degree of stress state triaxiality.

2. Gurson-Tvergaard-Needleman (GTN) Model

The description of the process of plastic fracture in metal alloys with technical applications is frequently presented using the model developed by A.L. Gurson [5], which was later modified by Tvergaard and Needelman. The model makes it possible to account both for the law of macroscopic plastic flow and the growth of micro-voids nucleating on inclusions and precipitations. The model application to the analysis of fracture process has been extensively investigated.

Void volume fraction is a basic parameter of the model. That is defined, in accordance with the formula below, as a ratio of a current micro-void volume to the sample volume:

$$f = \frac{V_V}{V} \tag{1}$$

where: V_v - current void volume fraction, V – sample volume

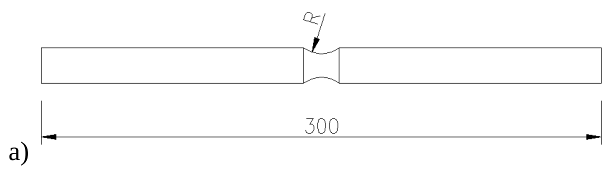
Strains and stresses are determined at macroscopic level. Gurson Tvergaard Needelman yield criterion is expressed as follows:

$$\Phi = \frac{\sigma_{eq}^2}{\sigma_M^2} + 2q_1 f \cosh \frac{3q_2\sigma_m}{2\sigma_M} - 1 - q_3 f^2 = 0 \tag{2}$$

where: Φ – non-dilatational strain energy, σ_{eq}- reduced stress in accordance with HMH criterion, σ_M - yield point, σ_m - mean stresses (the arithmetic mean of major stresses), q_1, q_2, q_3 – Tvergaard coefficients, f – current volume fraction of voids

3. Specimen Types and Investigation Methodology

In order to check how correct the computational results are and whether the GTN model parameters were selected properly, specimens with a ring notch underwent tension testing (Fig. 2a).



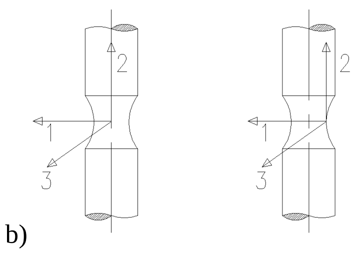


Fig. 2. Specimens used in investigations (a) and notation convention for individual components of strains (b).

The specimens used in investigations were 12mm in diameter, with the notch bottom radius R = 1, 4 and 10mm. Initial values of degree of stress state triaxiality in the axis of each specimen were equal to 1.943, 1.026, 0.670, respectively.

Numerical simulation of the tension tests performed on components with a notch was conducted. Computations were carried with ABAQUS Version 6.7 software. Standard axi-symmetrical CAX4R elements and the material GTN model were applied.

Computation results produced curves illustrating tension of components with a notch, graphs showing void volume fraction and values of strains accompanying the nucleation and rapid growth of micro-damages (in the specimen axis and on the notch surface). Notation convention for individual components of strains is presented in Fig. 2b.

4. Analysis of Results

The results of numerical computations for specimens with a notch are presented below. In all graphs, strain defined as the quotient of the specimen neck in the notch bottom by the initial diameter in this cross section, is an independent variable and a reference parameter.

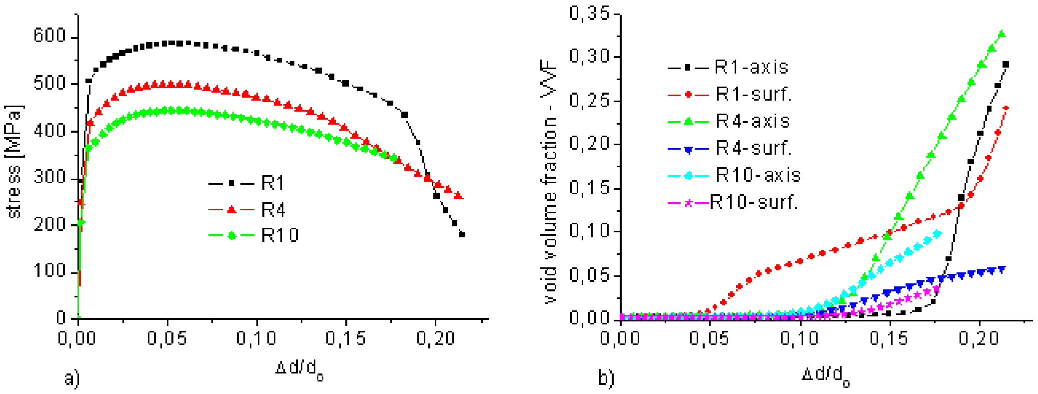


Fig. 3. Numerically determined tension curves (a) and void volume fraction in the axis and on the notch surface of specimens (b).

The analysis of the tension curve for specimens with 1mm notch (Fig. 3a) makes it possible to state that the stress maximum is reached when the strain $\Delta d/d_0$ equals 0.055. Fig. 3b shows that decreasing stress values are accompanied by a rapid increase in void volume fraction on the surface of the notch bottom, whereas the data in work [3] indicate void intensive development in the specimen axis. Rapid void growth was observed for the strain of 0.037. The rate of micro-damage development is relatively constant until the strain reaches approx. 0.19, after which the speed of the process increases rapidly again. At the instant of failure, the void volume fraction in the cross section amounted to 30%, which shows good congruence with the data in [3].

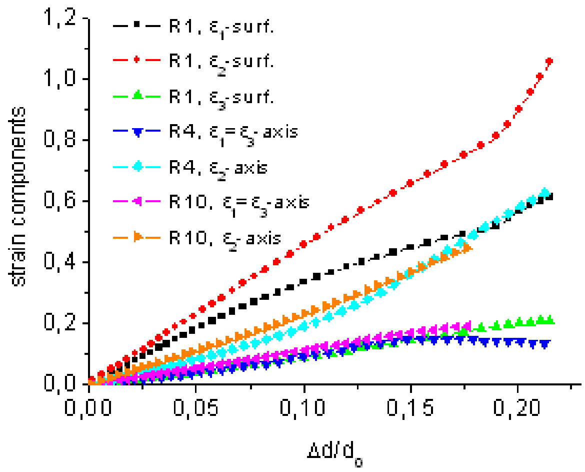


Fig. 4. Strain components in the areas of the highest void growth intensity.

Fig. 4 shows a change in strain component values computed for the point of the highest intensity of micro-void development at the strain initial stage. At the point corresponding to a rapid increment in void fraction (d/d_0 0.037), values of components ε_1, ε_2, ε_3 amounted to 0.13, 0.17 and 0.033, respectively. The process of micro-void nucleation occurred for strains of 0.011, 0.014 and 0.0014. The specimen failure took place for strain components that reached the values of 0.61, 1.06 and 0.21. The value of the effective strain $\varepsilon_{ef} = (\varepsilon_1^2+\varepsilon_2^2+\varepsilon_3^2)^{0.5}$ that accompanied void development amounted to ε_{efdev}=0.217, whereas the effective strain at the instant of failure ε_{effail} was 1.241.

Micro-damage development in specimens with the notch bottom radius 4mm demonstrates a slightly different character. The maximum in the stress graph is reached for the strain of 0.052 (Fig. 3a). Sudden growth in micro-damage volume fraction occurs a little later, after $\Delta d/d_0$ has reached the value of 0.11 (Fig. 3b). In contrast to R=1mm specimens, a more intensive development of micro-damage occurs in the specimen axis. At the instant of failure, the void volume fraction amounted to 0.33 in the specimen axis, and to 0.06 on the notch bottom surface. According to [3], the failure of a specimen of similar geometry (R=3.5mm) occurred when the void volume fraction amounted to 0.26, which shows a good congruence of the results obtained.

Fig. 4 presents changes in strain components in the specimen axis. In the case discussed, ε_1 and ε_3 are radial components, therefore the dependence $\varepsilon_1=\varepsilon_3$ holds (Fig. 2b). Strain components at void nucleation in the specimen axis amounted to $\varepsilon_1=\varepsilon_3$ =0.0005 and ε_2 =0.003. For the strain value d/d_0=0.11 (the start of the sudden micro-void development process), strain components at the point reached the values $\varepsilon_1=\varepsilon_3$=0.11, whereas ε_2=0.22 (the effective value ε_{efdev} 0.269). The specimen failed when $\varepsilon_1=\varepsilon_3$ =0.14, and ε_2=0.63 (ε_{effail} of the value of 0.66).

By conducting comparative analysis of results obtained for specimens R=4 and R=10mm, numerous analogies can be found. The strength of specimens with 10mm notch was obtained for the strain of 0.052 (Fig. 3a). Similarly to the previous case, the sudden development of micro-voids occurred when the material strength was exceeded, at the strain of 0.08 (Fig. 3b). It should be noted that the intensity of micro-void growth was decidedly higher in the specimen axis. At the instant of failure, the void volume fraction at that point amounted to 0.1.

The character of strain increment at the point (in the specimen axis) is similar to linear (Fig. 4). The dependence $\varepsilon_1=\varepsilon_3$ holds, like before. Strain components at void nucleation were $\varepsilon_1=\varepsilon_3$ =0.0008 and ε_2 =0.003, whereas at the onset of rapid micro-void growth, strain components were $\varepsilon_1=\varepsilon_3$ =0.09 and ε_2=0.19. At the instant of the specimen failure, the following values were obtained: $\varepsilon_1=\varepsilon_3$ =0.19 and ε_2 =0.45. Values of effective strains thus amounted to ε_{efdev} 0.229 at the instant of micro-void rapid growth, and to ε_{effail} 0,52 at the instant of failure.

5. Conclusion

Numerical computation performed with GTN model made it possible to identify the areas of rapid micro-damage development, and hence the sites of initiation of ductile fracture. It was found out that for a sharp notch (R=1mm), the failure process is initiated on the specimen surface. In specimens with relatively low degree of triaxiality (R=4 and 10mm), more pronounced micro-damage development occurs in the component axis. It was demonstrated that a decrease in the material strength is strongly related to the process of the material internal micro-damage development.

On the basis of analysis of strain components at the points of fracture initiation, it should be stated that no matter what the case discussed was, the components on the loading direction (ε_2) showed the highest values. Rapid development of micro-damage occurred at effective strain values ranging from 0.217, for specimens with 1mm notch, to 0.269, for 4mm notch.

References

[1] NEIMITZ, A. *Fracture Mechanics* (in Polish). Wydawnictwo Naukowe PWN, Warsaw 1998.

[2] GOLASKI, L. *Elements of Experimental Fracture Mechanics* (in Polish). Kielce University of Technology Publishing House, Kielce 1992.

[3] KOSSAKOWSKI, P., TRAMPCZYNSKI, W. *S235JR Steel Failure Numerical Simulation Taking into Account the Effect of Micro-structure Damage.* Przeglad Mechaniczny, paper approved for publication.

[4] NOWAK, Z. *Identification Method in Mechanics of Ductile Materials with Damages* (in Polish), habilitation work, Institute of Fundamental Technological Research, Polish Academy of Sciences (IPPT PAN), Warsaw 2006.

[5] GURSON, A. L. *Continuum Theory of Ductile Rapture by Void Nucleation and Growth Part I – Yield Criteria and Flow Rules for Porous Ductile Materials*. Journal of Engineering Materials and Technology, 1977, 99, pp. 2-15.

The publication is co–funded by the European Union from the European Social Fund, project: *Teaching Potential Development Programme at the Kielce University of Technology: Education for Success*. Human Capital Operational Programme, agreement no. UDA – POKL.04.01.01 – 00 – 175/08-00.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

Application of Georadar to Diagnostics of Prefabricated Slab Buildings

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Abstract. The paper discusses results of a georadar analysis conducted for a building of prefabricated reinforced concrete slabs. It aimed to determine the number and position of ties carrying the protective slabs in order to estimate the load carrying capacity of ties. The results obtained through the analysis were compared with the design data and used for strength calculation of ties carrying higher loads due to additional styrofoam layer.

Keywords: Reinforced concrete, prefabricated slab buildings, nondestructive testing, georadar.

1. Introduction

The technical condition of structures of prefabricated slabs has raised numerous concerns and been widely discussed. The analysis of the structures constructed in Poland in the years 1962-2004, conducted with respect to building technologies, shows that as much as 11% of collapses and structural failures occurred in prefabricated reinforced concrete structures, whereas 14% of failures and collapses happened in slab structures. As regards the function of damaged components, the analysis demonstrates that 5% of failures and collapses concerned ties [1].

Multi-leaf walling, which is characteristic of it, is composed of concrete load bearing leaf, heat insulating layer (of styrofoam or rock wool) and outer leaf made of concrete (Fig. 1).

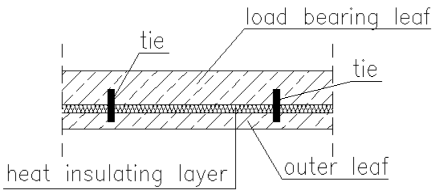

Fig. 1. Diagram of three-leaf walls of large panel buildings.

Metal tie bars (hangers and pins) constitute indispensable components of the large panel system. The possibility of ties being damaged poses a substantial threat, as it can result in the face panel falling off, which is particularly dangerous, as the failure is not preceded by any symptoms [2]. Such incidents already occurred in Kielce, Rzeszow, Gdansk, Starachowice and in Silesia.

2. Description of the Structure under Investigation

Georadar investigations were conducted on parts of the end wall of the students' house no. 2 of the Physical Education University in Cracow (Fig. 2).

In connection with the structure thermal upgrading, which meant outer panels and tie bars extra loading with insulation layers, it became necessary to check the load bearing capacity of the ties. The calculations were made by the upgrading designer at the assumption that ties were distributed in the structure in accordance with the W-70 system design data. In order to check the assumptions made (on the location of hangers), the designer commissioned to localise the ties with non-destructive testing. As a result, the structure investigations were conducted with georadar technique.

Fig. 2. End wall of students' house no. 2 of the Physical Education University in Cracow.

3. Georadar Method

Primarily, georadar has geological applications. Currently, the equipment is increasingly more frequently used in non-destructive road surface tests, and also in the diagnostics of building structures, mainly those of reinforced and prestressed concrete [3]. Georadar is one of electromagnetic diagnostic methods, based on the principle of wave emission (with a transmitter antenna) into the interior of an object under consideration. The parameters (mainly the amplitude) of the reflected wave recorded by the receiver are analysed. Those strongly depend on the dielectric properties of the material being examined.

The most typical georadar application includes the identification of different materials inside a medium, carried out on the basis of the analysis of the change pattern in the reflected wave amplitude. For reinforced concrete structures, that is the change in the amplitude of the wave reflected from the steel reinforcement bars against the background of waves reflected from the concrete matrix. The mechanism of generating signal from linear components (e.g. bars) is presented in Fig. 3. The antenna advanced over the component surface transmits a signal into the medium interior. The amplitude of the wave reflected from a "foreign" body (the medium nonlinearity) and recorded by the receiver radically differs from the "background". The radar image of a bar is generated due to wave sequence. Results visualisation can be made in many ways. The simplest is two-dimensional echograph, in which the horizontal axis represents the antenna position and the vertical axis – the wave return time. A change in the echograph hue intensity gives the information on the reflected wave amplitude. The bar radar mapping (echograph) takes on a characteristic form of an inverted hyperbola (Fig. 3)

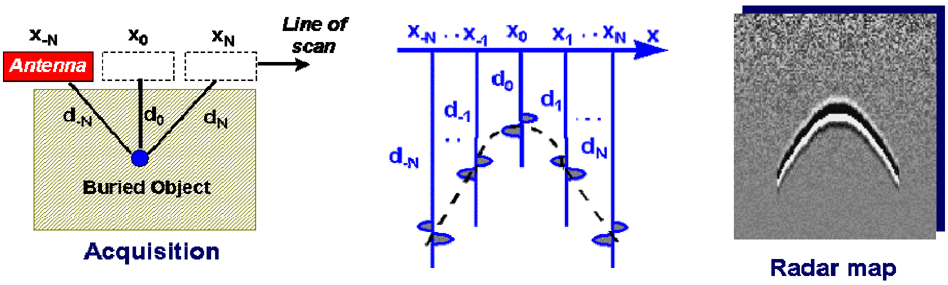

Fig. 3. Principle of georadar measurement [3].

4. Investigation methodology

Measurements were taken using Aladdin georadar manufactured by IDS company. The device was equipped with 2GHz bistatic antenna and had a range of 100 cm.

The investigations were conducted on the prefabricated end wall of the building under consideration (Fig. 4).

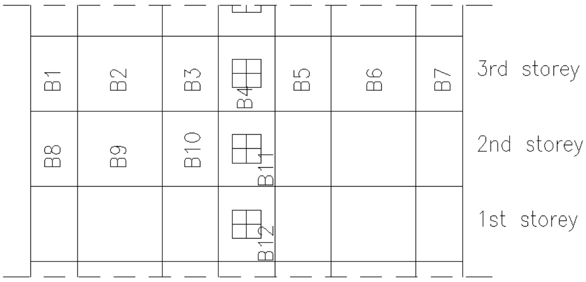

Fig. 4. Diagram of the building wall under investigation.

Four basic panel types were differentiated: three – module ones, 180 cm in width (denoted as B1, B7 and B8), 240 cm four – module ones (B3, B5 and B10), 360 cm six – module ones (B2, B6 and B9) and a four- module panel, 240 cm in width, with a window opening (B4, B11 and B12).

At the stage of interpreting the results, it was assumed that an additional signal (hyperbola) between signals from the regular mesh of reinforcement bars represented the image of the tie (Fig. 5).

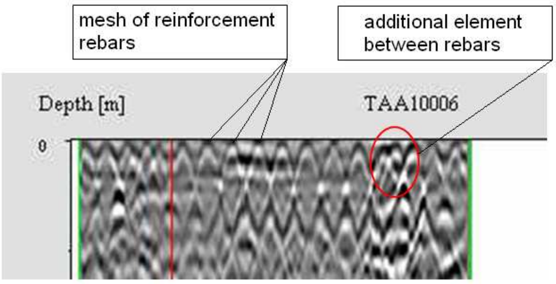

Fig. 5. Exemplary echograph recorded in measurements.

5. Results of Analysis

Fig. 6 shows exemplary results for selected panels of each type. Diagrams on the left side present the results of tests. Those on the right side represent the W-70 [4] system design data for individual panel types.

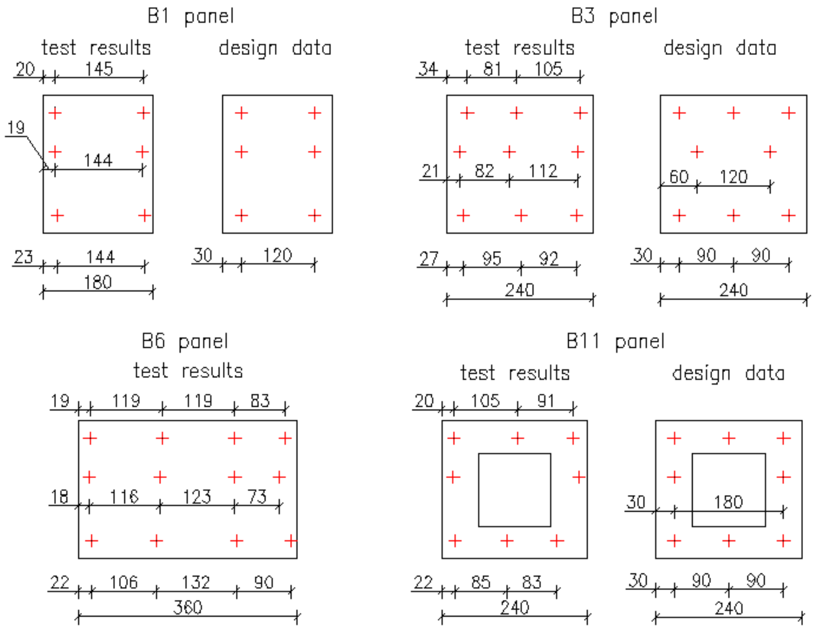

Fig. 6. Distribution of ties in individual panel types (determined due to investigations and in accordance with the design data [4]).

Results of tests for three – module panel were shown on the basis of B1 panel. In accordance with Fig. 6, georadar investigations made it possible to identify six ties spaced 144 and 145 cm apart. The number of ties located shows good congruence with the design data.

In four – module panels (B3 in Fig. 6), as far as the number of ties is concerned, congruence with the design data was shown for the upper and lower edges with the dimensional variation up to 15 cm. In the row located at the height of 170 cm, the tests revealed three ties, whereas the design data show only two of those.

The analysis of results for six – module panels (B6 in Fig. 6) demonstrated twelve ties spaced in three rows. Here the comparative analysis could not be made as the design data were not available.

The highest congruence of results was found for the four- module panels, with a window opening (B11). Ties were arranged in three rows following the pattern of 3-2-3 items. Differences in tie spacing, however, amounted to 15cm.

6. Conclusion

The results presented in the paper clearly indicate that the georadar method is a suitable tool to run diagnostic tests of large panel structures. The application of georadar made it possible to determine the location of hangers and pins that tie outer leaf panels. The results of tests show relatively good congruence with the design data. The differences observed, however, should not significantly affect the load bearing capacity of ties.

The georadar method was not useful for determining the diameters or a degree of corrosion advancement in the anchoring elements, which can significantly affect their load bearing capacity. The results obtained provide a basis to determine the sites, at which test pits could be made or other non-destructive methods might be applied to complete the data that are missing.

References

[1] RUNKIEWICZ, L. *On Structural Failures and Collapses in Poland in the years 1962-2004.* (in Polish). Inzynieria i Budownictwo, no. 4/2006.

[2] PLONSKI, W. *Durability of Buildings of Prefabricated Large Panels* (in Polish). Przeglad Budowlany, no. 10/1996.

[3] MANACORDA, G. *A Short Introduction to the Georadar Theory*. IDS company training materials.

[4] *W-70 System Prefabricated Panels Design Catalogue* (in Polish).

[5] WCISLIK, W. *Georadar Investigations into Bridge with Prestressed Concrete Deck Beams* (in Polish). Papers of 56 Scientific Conference of the Committee of Civil and Water Engineering of the Polish Academy of Sciences and the Science Committee of the Polish Association of Civil Engineers and Technicians, Krynica 2010.

The publication is co–funded by the European Union from the European Social Fund, project: *Teaching Potential Development Programme at the Kielce University of Technology: Education for Success*. Human Capital Operational Programme, agreement no. UDA – POKL.04.01.01 – 00 – 175/08-00.

Transcom 2011, 27-29 June 2011
University of Žilina, Žilina, Slovak Republic

The Analysis of Traffic Intensity at the Intersection on Łódzka and Transportowców Streets in Kielce

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Abstract. A measurement of traffic intensity at the crossing is performed in order to determine possible changes in traffic intensity value, changes in generic and directional structures or to update signal group diagrams. A survey is made at different hours, days of week and month to designate a rush hour.
A subject of research was a comparative analysis of traffic intensity at the junction on Łódzka and Transportowców Streets in Kielce in two days: on the 28th April 2010 (a common Wednesday) and on the 12th May 2010 (market Wednesday). The results of the research lead to a conclusion that, an organization of Targi Kielce and the traffic, which is generated by the participants do not have an influence on increase the traffic intensity value on the junction.

Keywords: Traffic intensity, intersection, comparative analysis.

1. Introduction

Traffic intensity is a value of a traffic flow or traffic stream observed under the given cross-section on the segment between the junctions or at the inlet crossing, which is expressed by the number of actual or conventional vehicles passing under consideration a cross-section per unit time.

The researches of traffic intensity have been performed to determine:

- whether and how much a value of the traffic intensity is changing depending on organized events at the Targi Kielce,
- changes in generic and directional structures.

A measurement of traffic intensity is made above all during peak periods, which fall on different hours of days, depending on the location of a crossing and character of the movement on the intersecting or connecting roads [1, 3].

In choosing the term of measurements, there have been included:

- recommended periods execution of traffic count for crossings located on built-up area of city, according to which a survey should be carry out in months: April, May, September, October, in working days from Tuesday to Thursday, between the hours of 6:00 am to 9:00 am and 2:00 pm to 6:00 pm [2].
- a timetable of exhibition events and opening hours Targi Kielce for visitors.

A basic interval of registration vehicles is one hour. For special purposes (for example estimation capacity) it is used interval: 30, 15, 5 minutes.

Research is performed in several ways:

- manual record,
- using automated counter,
- using a film camera [1, 3].

2. The Subject of Research

The object of research is an intersection on Łódzka and Transportowców Streets in Kielce. This is a four-entry crossing situated in northwest part of the city. Łódzka Street is a single-roadway, bi-directional street which is a part of national road 74 (Sulejów – Kielce – Zosin – Polish border). In addiction, this is the road, which leads traffic into the city and out of Kielce, serves a through traffic on the relation Kielce – national road S7 and it leads the traffic from the centre of city to exhibition area of Targi Kielce which is located on Zakladowa Street. This situation has been illustrated in Fig. 1. Transportowców Street and service street fulfil a function as access roads. Movement of vehicles and pedestrian traffic on the analyzed intersection has been operated since 2000 by the vehicle actuation, tri-group diagram and tri-signal phase. Scheme of the analyzed intersection at Łódzka and Transportowców Streets has been shown in Fig. 2.

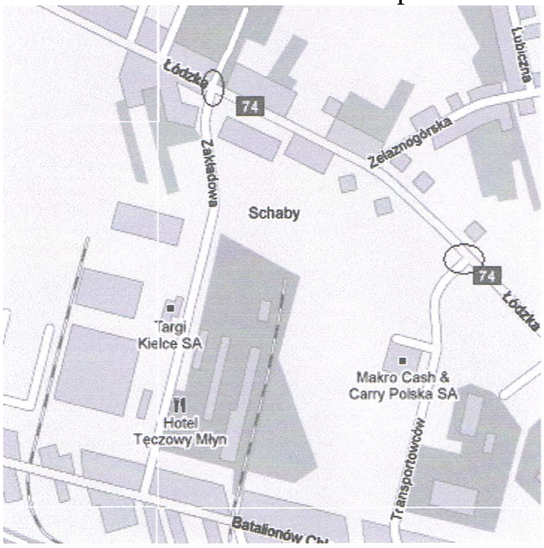

Fig. 1 Location of the analyzed intersection at Łódzka and Transportowców Streets in Kielce's transport system [5].

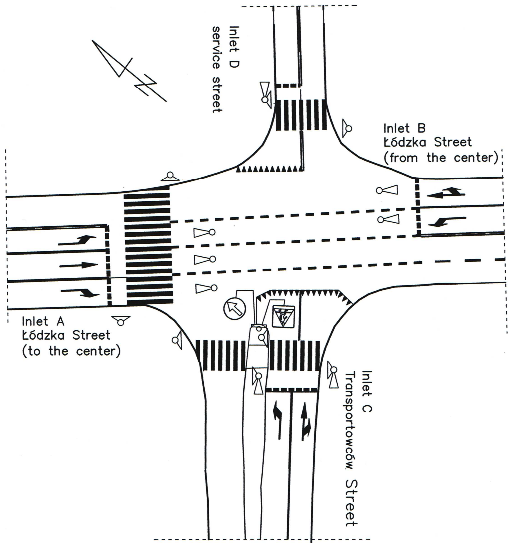

Fig. 2. Scheme of the analyzed intersection at the Łódzka and Transportowców Streets in Kielce [4].

Over the past years it has been observed the increase of traffic intensity on this crossing, especially after building self-service warehouse MAKRO Cash & Carry in 1999 adjacent to analyzed intersection. Otherwise, near to the inlets of the intersection many service and industrial establishments are situated.

3. A Measurement of Traffic Intensity at the Intersection

The researches of traffic intensity on the inlets of the analyzed crossing were done twice: on the 28th of April 2010 (a common Wednesday) and on the 12th of May 2010 (on the second day of Targi Kielce – AUTOSTRADA-POLSKA, TRAFFIC-EXPO-TIL, MASZBUD on Wednesday). The idea of the survey was a comparative analysis of traffic intensity on a crossing during the day with typical traffic and with the traffic generated by visitors of Targi Kielce. The warehouse is located near to the discussed crossing of streets: Łódzka - Transportowców.

Researches have been executed by every vehicle passing through the inlet of the intersection direct registration. Measurements were done in two peak periods:

a) common Wednesday – the 28th April 2010
from 7:00 am to 10:00 am
from 1:30 pm to 5:30 pm
b) market Wednesday – the 12th May 2010
from 9:00 am to 11:00 am
from 2:00 pm to 5:30 pm.

On the grounds of summary the registered traffic intensity a Figure 3 has been carried out, which illustrates traffic fluctuation in 15-minutes interval in both measuring days. Traffic intensity (Q) is given in vehicles per hour (v/h).

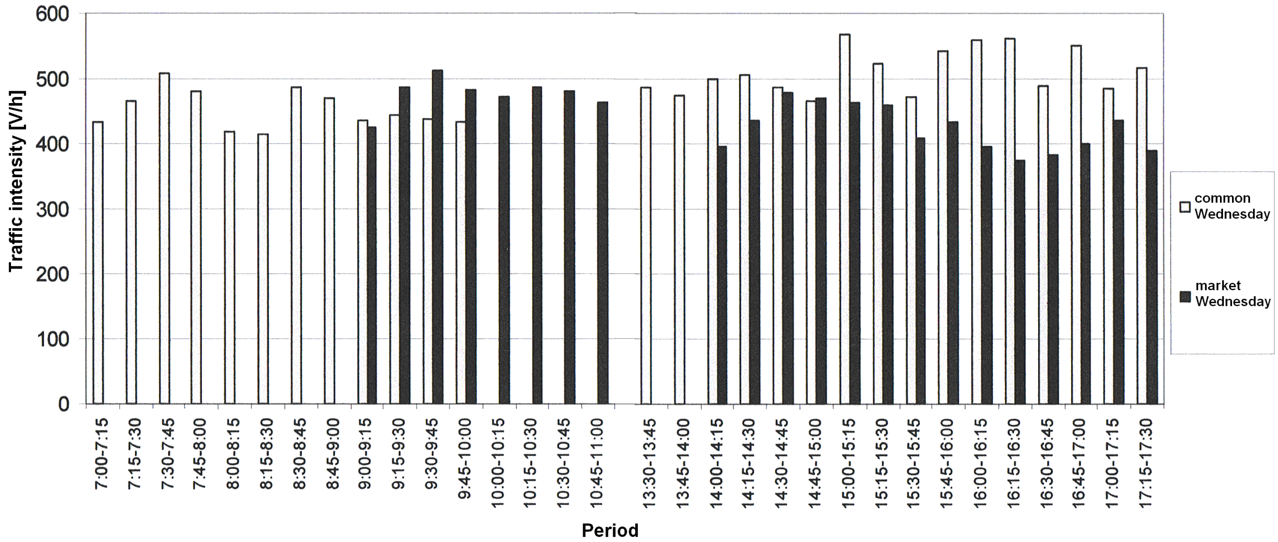

Fig. 3. Comparative summary traffic fluctuation in 15-minutes interval on junction in both measuring days [on 28th of April 2010 – common Wednesday and on 12th of May 2010 – market Wednesday].

A diagram shows that traffic intensities during the morning peak period were comparable, it oscillated around 400 – 500 vehicles per hour. Considerable differences were observed during the afternoon peak. Values of traffic intensity of common Wednesday and market Wednesday varied from each other even about 200 vehicles per hour.

On the basis of complete datasheet from performed research and executed analysis [4], there were designated morning and afternoon rush hours. For the purpose of stressing differences between traffic intensity during rush hours in both days there were prepared cartograms in passenger car unit (PCU) per hour, which is shown in Fig. 4. and Fig. 5.

The result of this comparative analysis is surprising. It turned out that significant impact on big difference in traffic intensity values has got just one relation on one inlet (Łódzka Street from the centre).

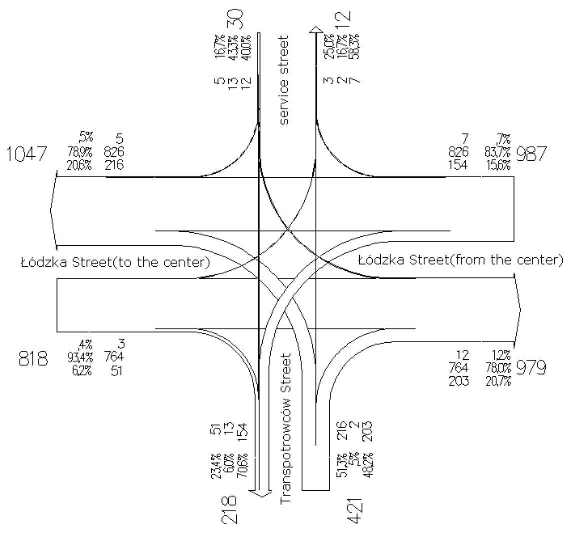

Fig. 4. Cartogram of traffic intensity in the afternoon rush hour [4:00 pm – 5:00 pm] on the 28th April 2010 [common Wednesday] [4].

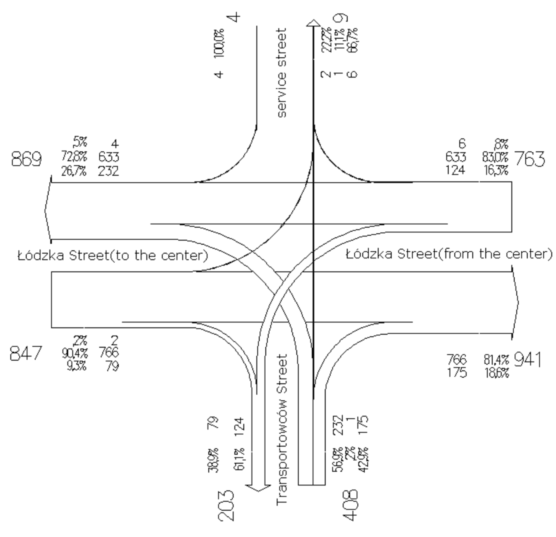

Fig. 5. Cartogram of traffic intensity in the afternoon rush hour [2:30 – 3:30] on the 12th May 2010 [market Wednesday] [4].

4. Conclusion

The following conclusion can be made on the basis of the conducted researches of traffic intensity at the crossing on Łódzka and Transportowców Streets in Kielce:

- fundamental difference is the period of a day, where the morning and afternoon peak hour has been occurred
  a) common Wednesday (the 28th April 2010):
    - morning rush hour: 7:00 – 8:00 am,
    - afternoon rush hour: 4:00 -5:00 pm;
  b) market Wednesday (the 12th May 2010):
    - morning rush hour: 9:15 – 10:15 am,
    - afternoon rush hour: 2:30 – 3:30 pm;
- directional structures at the individual inlets of intersection at Łódzka and Transportowców Streets has not gone under significantly differences, taking into account a day of measurement (common or market Wednesday). Predominant relations at the junction are: straight on the inlet Łódzka Street (to the center) and straight on the inlet Łódzka Street (from the center). The sum of traffic intensity at all of intersection was equal 2256 PCU/h (common Wednesday) and 2022 PCU/h (market Wednesday). Difference of about 200 vehicles has occurred just on one inlet – Łódzka Street (from the center);
- during the afternoon period the traffic intensity at junction on market Wednesday was much smaller comparing to value of traffic intensity on common Wednesday, despite it was an aggregate vehicles of typical Road users and visitors of Targi Kielce. No doubt the intersection at Łódzka and Zakładowa Streets, which is situated in the vicinity of the analyzed junction at Łódzka and Transportowców Streets in Kielce has got an affect on that results. Location of both crossings is portrayed in Fig. 1. The most of the Targi Kielce participants have headed towards the intersection of Zakładowa Street and Łódzka Street. High-impact vehicle on this inlet, during the afternoon peak period , it means in hours of the end of Targi Kielce exhibitions has caused an effect of blocking this intersection on Łódzka and Transportowców Streets by neighboring crossing. The result of this situation was significant enlargement of maximum and residual queue length at the analyzed intersection as well as impossibility in passing crossing in a smooth manner, what influenced the reduction of value traffic intensity at analyzed intersection.

References

[1] GACA S., SUCHORZEWSKI W., TRACZ M., *Inżynieria ruchu drogowego*, Warszawa 2008.

[2] Wytyczne projektowanie skrzyżowań drogowych. Część I – Skrzyżowania zwykłe i skanalizowane, Warszawa 2001.

[3] MAJOR H., WAWRZYNKIEWICZ B., BĄDEL B., *Inżynieria ruchu drogowego, część II - Materiały pomocnicze i ćwiczenia*, Kielce 1983.

[4] WYCHOWANIEC A., *Ocena warunków ruchu na skrzyżowaniu ulic: Łódzka -Transportowców wraz z koncepcją zmian w organizacji ruchu,* Written under the academic supervision of PhD. Eng Halina Major), Kielce University of Technology, 2010.

[5] www.maps.google.pl.

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TRANSCOM 2011

Proceedings, Section 7

Published by University of Žilina
First Editions
Printed by EDIS-Žilina University publisher

ISBN 978-80-554-0376-2