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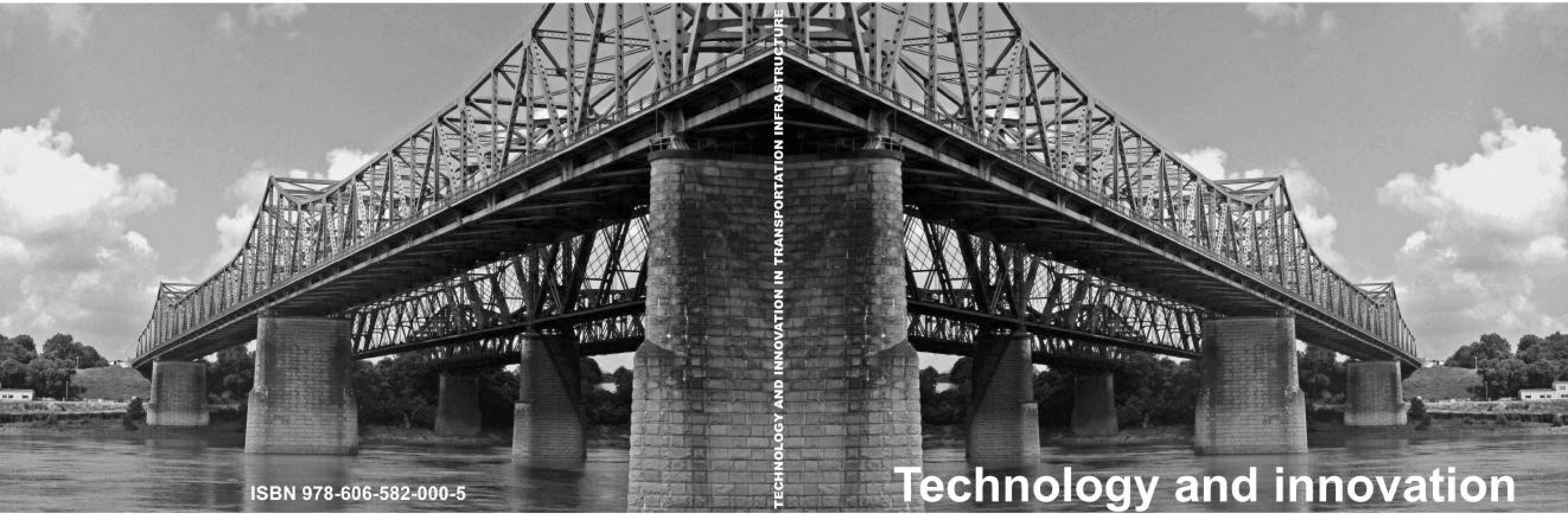
Rodian Scînteie
Costel Pleşcan
editors



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TECHNOLOGY AND INNOVATION IN TRANSPORTATION INFRASTRUCTURE

Technology and innovation in transportation infrastructure

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Rodian Scînteie, Costel Plescan
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Lightweight High-Strength Concrete

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Summary

In the design of bridge structures the material aspect is very important. Material design and its appropriate use in the construction have a major impact on its usability and durability. Concrete is the most commonly used material in bridge engineering. Most of the existing bridges were designed many years ago and by today’s terminology are from normal-weight concrete. This concrete has its own specifics. It is inexpensive, easy to technological discipline and builders have experiences with it, so they know what to expect from him. On the other hand, they have lower durability and strength, which is for challenging structures such as bridges inadequate. This disadvantage can be easily eliminated by using high-strength concrete (HSC). According to ordinary concrete, it has higher durability and the strength in particular, making it ideal for sophisticated constructions. It is thus possible to build more subtle structures in compliance with the strength requirements defined by the structural engineer. The initial price is higher than that of ordinary concrete, but ultimately, due to higher life-cycle and lower maintenance costs and redevelopment, they comes out economically much better than ordinary concrete. Their applicability has been demonstrated on a number of structures.

The presented article is devoted to high-strength lightweight concrete, which is a special group of HSC with low volume weight. Their advantage is further lightening the structure while the strength is maintained at a very good level.

KEYWORDS: Light-weight concrete, high-strength concrete, LWHSC

1. INTRODUCTION

Research on the construction materials field over the last decades has undergone significant changes and progress which is evident in concrete technology, mortar and ceramics particularly. Especially the concrete technology has advanced significantly ahead. New arts of concrete have been developed and manufacturing processes improved. The main forces in this way were concrete power such as

Japan, Norway, USA, Canada, Germany. Here were laid the foundations of new concrete arts such as high-strength, self-compacting, fibre, reactive powder, light high-strength and other concretes. Of extraordinary importance are mainly self-compacting (SCC) and high strength (HSC) concretes. SCC makes easier concrete storage into the formwork and increase working efficiency by eliminating the need for mechanical vibration. In this point of view, staff and costs are reduced. HSC enables to build constructions demanding strength like bridges, high-rise buildings, military objects, dams, etc. The intelligent design also saves money, because as compared with ordinary strength concrete they have higher strength. Thereby more subtle constructions can be built, usable area of the construction increases and it all together saves material. Their very important feature is the durability, which significantly extends the life of HSC structures. By the HSC possibility to build more subtle constructions, it can be achieved significant lightening of the structure, which is interesting for example in bridge engineering, where it can be designed longer spans. Very interesting in this regard is then lightweight high-strength concrete (LWHSC), which by maintaining high strength significantly reduce constructions own weight (dead load). As is known, lightweight concrete is relatively used material. Application founds at strength unassuming structures, thermal insulation structures, but even where it is necessary to reduce concrete structure mass. Light HSC finds its application only. Yet already it has been used in the construction of bridges (Sandhornoya Bridge in Norway, The Sebastian Inlet Bridge in Florida), high rise buildings (The North Pier Tower in Chicago), marine structures (Hibernia offshore platform, Heidron floating concrete platform) or older buildings renovations. [1, 2]

Figure 1. Sandhornøya bridge [3]

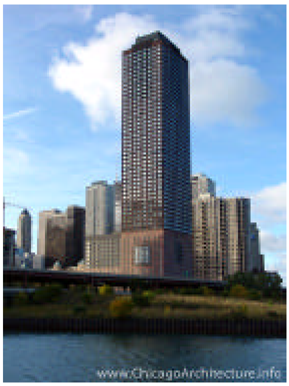

Figure 2. North Pier Tower, Chicago [4]

Figure 3. Heidrun Tension Leg Platform [5]

2. LIGHTWEIGHT CONCRETE

Lightweight high-strength concrete was developed from a lightweight concrete. Low bulk density is achieved by using lightweight aggregate. Increased strength by higher cement doses, effective mineral and chemical admixtures addition and using strong light aggregate. According to CSN EN 206-1 the lowest LWHSC class is the LC 55/60. After 28 days of curing, lightweight concrete must to have compressive strength at least 60 MPa. But many experts around the world counts among LWHSC concrete with compressive strength of 50 MPa after 28 days of curing too.

Lightweight concrete using beginnings dates back to antique times. In this context the Pantheon is often mentioned. The dome was built of lightweight concrete (less known is then lightweight concrete use in port constructions). The frame consisted of lightweight volcanic aggregates. Then the lightweight concrete was for centuries forgotten. The rediscovery and re-use began to occur in the early 20th century with concrete technology development. Today lightweight concrete is a standard material. Lightweight aggregate is usually artificially burned.

Lightweight high-strength concrete is the most modern art of lightweight concrete. It combines the advantages of lightweight and high-strength concrete. It is a very specific material. Its design needs together with experience and practical knowledge mainly high-quality lightweight aggregates. This is because in this type of concrete, the weakest link in terms of strength. Commonly used lightweight aggregates generally do not achieve higher compressive strength than 50 MPa. Production of such high-strength concrete therefore requires a very high quality aggregate. If available, really impressive concrete parameters can be made.

Compressive strength of 100 MPa measured on cubes with the edge of 100 mm have already been made experimentally. Eg. Berra and Ferrara reached 60 MPa and bulk density of 1 700 kg·m^{-3} with 150 mm cubes. [1, 2]

The strength of LWHSC depends not only upon the density and strength of lightweight aggregate. There are many factors such strength of mortar matrix, bond between the aggregates and the matrix, concrete density, age, water to binder ratio, degree of compaction, curing, etc. The final strength of LWHSC depends on those and other factors. Moreover, tests have shown that the rule „more cement higher strength" does not apply. There are many reciprocal influences and combinations. It is understandable that not all problems have so far been satisfactorily explained and understood. Sometimes till the trial batch shows the real strength.

2.1. Raw materials

To achieve the desired parameters of concrete it is necessary to choose optimum type and quantity of raw materials. The most important is the choice of lightweight aggregate, which plays the major role in LWHSC. Appropriate cement and admixtures must be also used too.

2.1.1. Lightweight aggregate

Lightweight aggregate is indeed expensive, but allows concrete density reduction.

It may be of natural origin:

- Volcanic (pumice, scoria)
- Organic (palm oil shells, shiv, rice husks)

or synthetic:

- Natural materials (perlite, vermiculite, clay)
- Industrial products (glass)
- Industrial by-products (fly ash, expanded slag cinder)

The most known types of lightweight aggregates are Liapor (made from keramsit clay) and Lytag (made from fly ash). Because of its high water absorption, the wetting is recommended before dosing in concrete. When designing the concrete composition it is also necessary to calculate the aggregate bulk density instead of weight. The density of LWAC varies from 50 kg·m^{-3} for expanded perlite to 1 000 kg·m^{-3} for clinkers and varies for the same type of aggregate too. [2]

Table 1. Some dry densities of lightweight aggregates [2]

Aggregate type	**Dry density** $kg{\cdot}m^{-3}$
Clinker	700 – 1 000
Sintered pulverized ash	750 – 950
Expanded clays and slag	300 - 950
Liapor	500 - 950
Lytag	750 – 1 100
Pumice	500 – 900
Scoria	800 – 1 000
Palm oil shell	550 – 650
Expanded perlite	30 - 240
Expanded vermiculite	60 – 190

2.1.2. Superplasticizers

High-strength concrete is made with high content of cementitious materials. Its bigger specific surface needs to be balanced by adding high range water reducers. They maintain good workability of concrete and homogenous dispersion of fine particles. As well as, reducing the water amount and thus increasing the strength. In recent years mainly acrylic polymers are used to. In contrast to older sulfonated naphthalene condensates and sulfonated melamine formaldehyde condensates they are very active for low water-cement ratios. Using superplasticizers care should be taken to the compatibility with some cements and to the dosage.

2.1.3. Binder

The low strength of the aggregate can be balanced by using high strength cement mortar and highly reactive pozzolan, such as silica fume. The mostly commonly used is the Portland cement 42,5 and 52,5 in the amount of 400 – 600 $kg{\cdot}m^{-3}$. Highly active mineral admixture such as microsilica or metakaolin will be added to the cement. Dosing is usually up to 10 % of cement weight. Higher amount is already uneconomic and the strength increase in lightweight concrete is very small. The silica fume is advantageous not only in terms of strength but also the mixture cohesion. Its disadvantage is the high price.

2.2. Lightweight high-strength aggloporite concrete

LWHSC properties may be illustrated on a research based example conducted on the artificial (synthetic) fly ash aggregate known in Czech Republic as aggloporite.

2.2.1. Aggloporite

Aggloporite is being self-burned on sinter grates. By burning it comes to sintering, combustible matter is burning up and typical porous structure will be formed. Aggregates properties are similar to other synthetic aggregates and are given mainly due to entering raw materials composition and through burning process run. Aggloporite grains porosity is usually around 50 %, void fraction depending on the crushing and sorting method about 50 – 60 %. The bulk density of aggloporite sand ranges from 600 to 1 050 $kg \cdot m^{-3}$, the coarse fractions 400 to 700 $kg \cdot m^{-3}$. Water absorption is very different too, usually 8 – 30 weight %. Thermal conductivity of aggloporite in free strewed state is 0,12 – 0,16 $W \cdot m^{-1} \cdot K^{-1}$. Higher quality aggloporite sorts cylinder strength is 2 – 8 $N \cdot mm^{-2}$.

2.2.2. Aggloporite concrete proposal and testing

When designing a suitable formula, several aspects were evaluated. In particular, these were: low aggregate strength, binder quantity for necessary strength achieving, economic performance and resulting bulk density. Soaked aggregates was dosing.

From several tested formulas it was chosen that listed in Table 2. This formula did not show the highest strength, but was more optimal from the price – performance point of view. For higher strength achieving the dose of silica fume should be increased and water-cement ratio decreased (for example). However, this approach is not very effective. Water-cement ratio reduction gets worse workability and the silica fume increase will increase the strength only slightly. The concrete is not able to handle the increased load due to low aggregate strength.

Table 2. Designed LWHSC formula

Component	**Amount in 1 m^3**	**Unit**
Cement CEM I 42,5 R	450	kg
0 – 4 mm Žabcice	680	kg
4 – 8 mm Olbramovice	140	kg
Aggloporite 4 – 8 mm	167	l
Aggloporite 8 – 16 mm	576	l
Mikrosilica	23	kg
Superplasticizer	8,3	w. %
Water	118	l
w	0,28	-

Fresh concrete slump was 200 mm at a bulk density of 1 970 $kg \cdot m^{-3}$. For concrete parameters determination the following tests were performed:

- compressive strength determination
- tensile strength determination

- static and dynamic modulus of elasticity determination

The main tests were carried out at the age of 28 days. Besides this, the strength development over time and form factor (influence of body size) was observed. Measured values are shown in Figures 4-7.

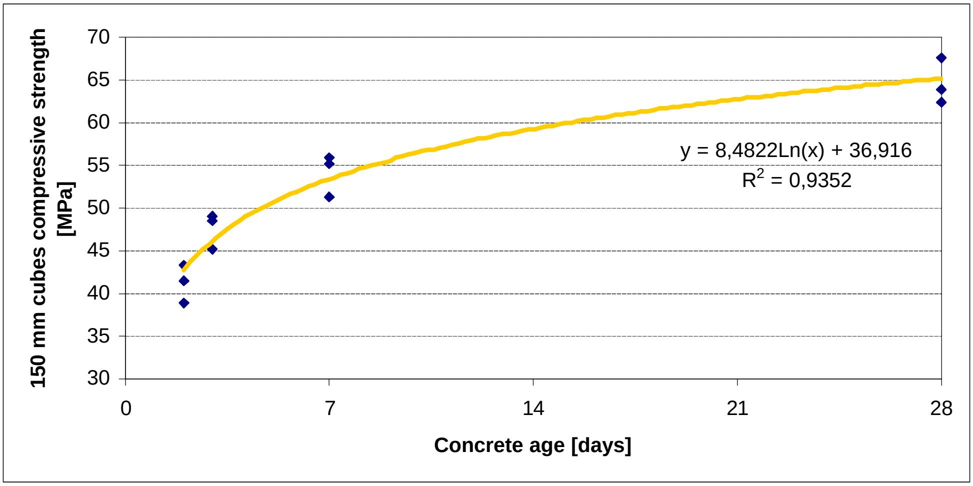

Figure 4. Compressive strength development in time

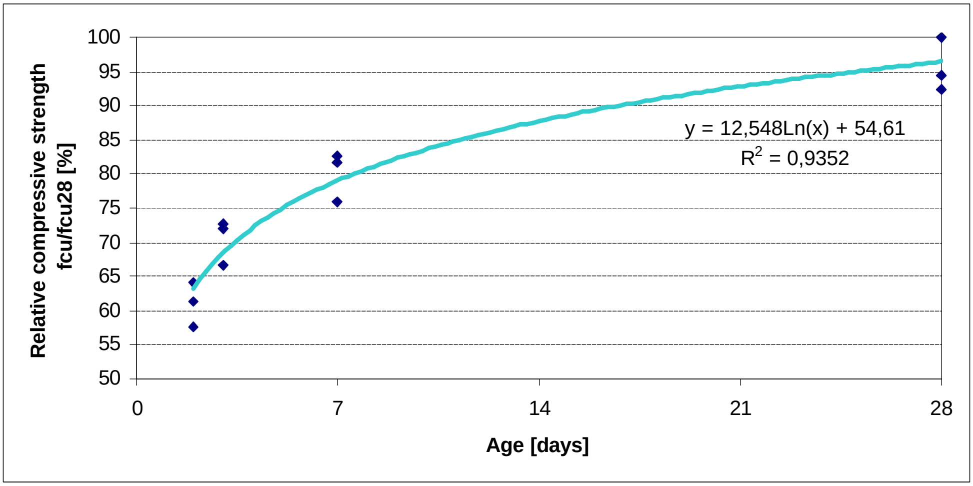

Figure 5. Relative compressive strength development in time

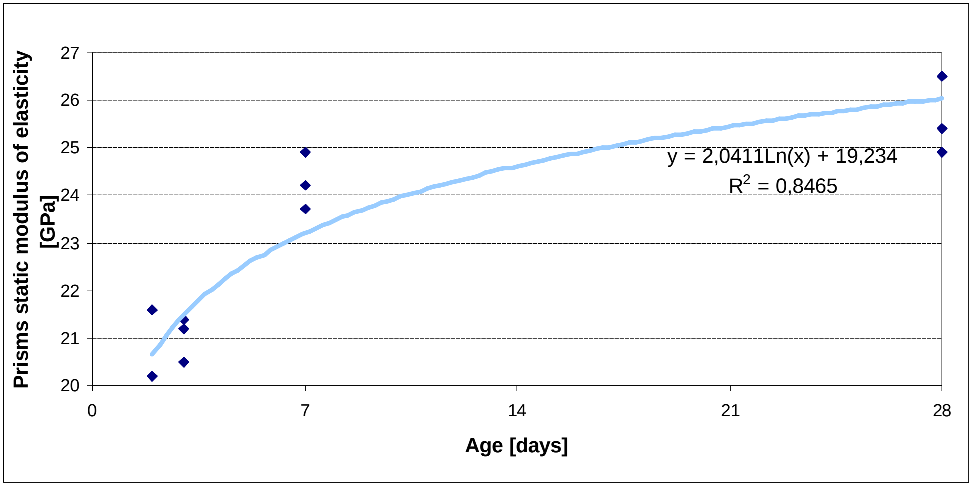

Figure 6. Temporal static modulus of elasticity development

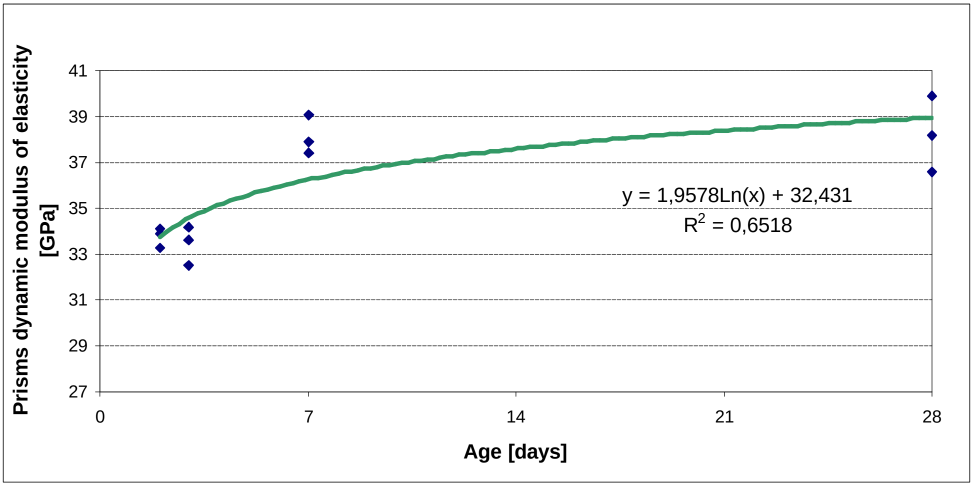

Figure 7. Temporal dynamic modulus of elasticity development

Table 3. Form factor

Cube size		
50 mm	100 mm	150 mm
0,68	0,77	1,00

Results shows, that by using of effective admixtures, higher cement content and high-quality mineral admixtures it is possible to produce high quality lightweight concrete. The designed concrete showed after two days strength around 42 MPa with the density under 1 950 kg·m^{-3}, which is quite good result for lightweight concrete. The static modulus is lower than it would be in the same class of concrete with normal aggregate. However, it is higher in comparison with the lightweight concrete of lower strength class. This is probably due to higher cement and silica dosage and denser structure. Dynamic modulus shows the greatest dispersion and low correlation coefficient r, which is 0,807. This is due to the high porosity of the aggregate.

Form factor is very different from ordinary concrete, where the strength ratio between 150 mm cube and 100 mm cube is approx. 0,95. These values should however be treated with caution because for the exact form factor determining tens of values are needed.

3. CONCLUSIONS

This paper shows some results obtained in research on high-strength lightweight concrete, where the power plant fly ash was used.

The obtained values can be seen that the lightweight concrete of higher strength may be applicable when taking into account strength/bulk density ratio. 150 mm concrete cube strength reached after 28 days approximately 65 MPa having 1 970 kg·m^{-3} the bulk density. This is indeed an interesting result, taking into account that high-strength granodiorite concrete would have problems to exceed 90 MPa with similar design formula and much higher bulk density. When using higher-quality lightweight aggregate, the achieved strength could be inherently higher.

LWHSC may be very valid material, as proved by its successful use in a number of foreign structures.

Acknowledgements

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Use of recycled materials in pavement construction

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Summary

By its Directive 2008/98/EC, the European Commission requires EU Member States to develop a waste management system which aims to minimize negative environmental effects. This paper presents two materials (recycled aggregates and recycled steel fibers) that currently, in some countries, are treated as waste. The economic value of these materials can be recovered by using to the construction of rigid pavement.

The cement concrete from construction and demolition works is a material with high economic potential value that is often sent to landfill. Adequate economic value of the demolished cement concrete can be obtained by recycling processing it into recycled aggregates witch can substitute natural aggregates.

The use of recycled aggregates in new construction works helps protecting the environment by preserving natural resources and reducing the construction and demolition waste storage. Recycled steel fibers are obtained by recycling of post-consumed tires, the benefits of use in the construction of rigid pavement being demonstrated by the recent FP6 STREP EcoLanes Project.

KEYWORDS: recycled aggregates, recycled steel fibers, post-consumed tires, rigid pavements.

1. INTRODUCTION

Since November 2008 the European Union has a new legislation on waste, Directive 2008/98/EC, which includes objectives to be achieved by 2020 by the Member States. The aim is to implement measures that will minimize the negative effects of generation and management of waste on the environment and human health. The high volume of works in civil engineering, industrial, hydraulic, transport, etc. lead to intensive and extensive exploitation of alluvial deposits from river beds, changing the landscape by exploiting various rocks from mountains, but also to the generation of large quantities of waste that pollute the environment and require storage. Construction and demolition waste (CDW) is generated to construction/renovation of buildings (by excess supply, materials/parts damaged, cutting of parts etc.), demolition (due to natural disasters, accidents, war, civil conflicts, vandalism etc.), construction/rehabilitation of roads and railways infrastructures and others.

The main material resulting as waste from construction/rehabilitation processes is cement concrete from which, by recycling, result recycled aggregates with the aspect of crushed aggregates that can be successfully used in construction/rehabilitation of pavement. Aiming to reduce consumption of natural aggregates (resource that, by overexploitation, is dramatically decreasing) some countries in Europe reach a level of 95% recycling of CDW.

Production and use of recycled aggregates in pavements meets advantages in terms of environment impact by reducing both CDW from landfills and consumption of crushed/quarry natural aggregates. This technology meets also difficulties such as lower cost of natural aggregates, cost of collection-processing-transport operations, lack of selective collection/storage of CDW, lack of advanced studies in understanding of recycled aggregates, variability regarding the quality of CDW, possible need of a special equipment for obtaining recycled aggregates with superior physico-mechanical characteristics.

In America, since 1982, ASTM included recycled aggregates resulted from crushed cement concrete into the crushed aggregate category.

2. RECYCLED MATERIALS

Materials under study are recycled aggregates from construction and demolition waste and recycling steel fibers derived from recycle of post-consumed tires.

On global scale, various researches have been completed regarding the recycling of cement concrete and use of recycled aggregates to the construction of new buildings. The ongoing research program “Innovative technologies and logistic solutions for use of construction and demolition waste to the construction of cement concrete and steel fiber reinforced cement concrete pavements” aims to develop both: the recycling technology of CDW in order to obtain recycling aggregates with superior physical and mechanical characteristics and guidelines for the design of rigid pavements with recycled aggregates. The high level of confidence of the results will be provided through laboratory studies/tests of materials, but also through accelerated testing of some experimental sectors on natural scale.

The steel fiber reinforcement technology of cement concrete and roller compacted cement concrete (RCC) pavements have been recently investigated and confirmed as reliable in the European research FP6 STREP EcoLanes Project "Economical and sustainable pavement infrastructure for surface transport".

2.1. Recycled aggregates

Recycled aggregates are obtained by crushing of the cement concrete (figure 1) demolished from buildings, bridges, foundations, pavements etc.

Figure 1. Typical demolition site of a building: a – demolition of the cement concrete structure of the construction; b – crushing of the demolished cement concrete; c – storage of the obtained recycled aggregates

It is recommended that, before demolition of the building, other than cement concrete elements to be removed as much as possible. The crushing operation of the cement concrete should be preceded, where appropriate, by removal of reinforcements and other embedded materials.

The handling and storage of both demolished cement concrete and recycled aggregates should be undertaken with care to prevent contamination with dirt and other types of waste materials (plaster, plastic, glass, wood etc.) or asphalt from pavements.

Documents/drawings from the execution of the demolished construction, especially those detailing the quality and composition of the concrete used, are of particular significance because they contain information on physical and mechanical characteristics of the cement concrete, information that can be useful in determining the recycling potential of the building.

The organization of a demolition site and equipment needed for the production of recycled aggregates are not much different than a site engaged in the production of crushed aggregates, therefore, the same equipment used in the process of conventional crushed aggregates is used for crushing, sorting and storage of recycled aggregates.

Crushing of cement concrete can be achieved in two ways: crushing by compression, crushing by impact (figure 2).

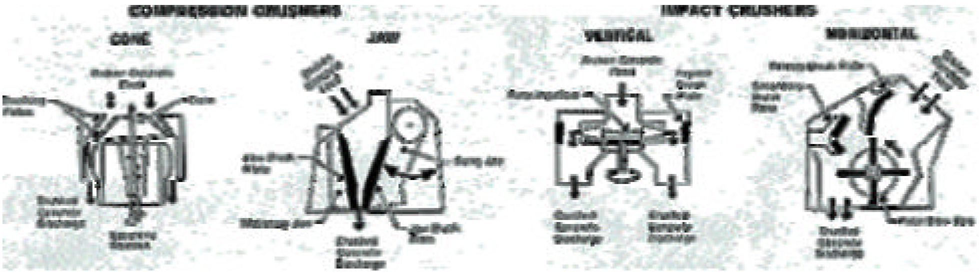

Figure 2. Description of crushing by compression/impact equipment

For better control of the grading of recycled aggregate it is recommended that the recycling process of the cement concrete to be achieved through two stages of crushing as shown in figure 3. In the primary stage, using a jaw crusher, the cement concrete is reduced to a granular mixture with 31…40mm maximum size particles. In the second stage of crushing, the material larger then 16/20mm is sent to an impact crusher where the desired maximum size of the recycled aggregate results.

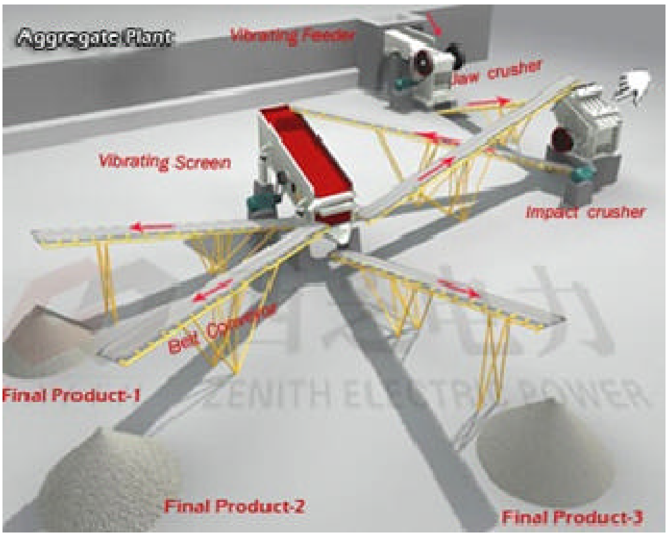

Figure 3. Technological description of cement concrete recycling in two stages of crushing

Typically, the operation of recycling the cement concrete has a yield of 75% coarse recycled aggregates and 25% of fine recycled aggregates. If the desired maximum size of recycled aggregate is smaller, for example 25mm, the yield of coarse recycled aggregate decreases.

2.2. Recycled steel fibers

Increasing and large quantities of used tires involve problems regarding the environment due to required storage space and high risk of fire as shown in figure 4.

Figure 4. a) storage of used tires; b) fire in used tires landfill

Tire recycling technology involves decomposition of tires into various of components, namely: rubber (75…90%), textiles (0.5…21%) and metal (3…15%).

Rubber resulting in tire recycling can be used in different applications as presented in figure 5 namely: railway antivibrational items (a), road safety items (b), flooring for playgrounds (c), thermo-acoustic insulation panels (d) and others.

Figure 5. Areas of use for rubber from used tire recycling

Although the recycled rubber can be used in a wide range of fields, as noted, the same cannot be said of steel fibers from used tires recycling process, these fibers being treated as waste.

The purpose of the EcoLanes research program was economic recovery of recycled steel fibers by use in rigid pavement construction. The main objective set in this program have been achieved through studies and laboratory tests on materials, but also by performing experimental sectors (scale 1/1) on NR17 Suceava – Vatra Dornei and on the accelerated testing facility ALT-LIRA from the Faculty of Civil Engineering of Iasi (for more details see [1], [2], [3], [4]).

The technologies for recycling of used tires, [1], can be:

- mechanical recycling processes by shredding or shearing after freezing;
- chemical recycling processes by pyrolysis or microwave induced pyrolysis;

Recycled steel fibers used in EcoLanes research program ware obtained by shearing process of used tires, the recycling technology is highlighted schematically in figure 6.

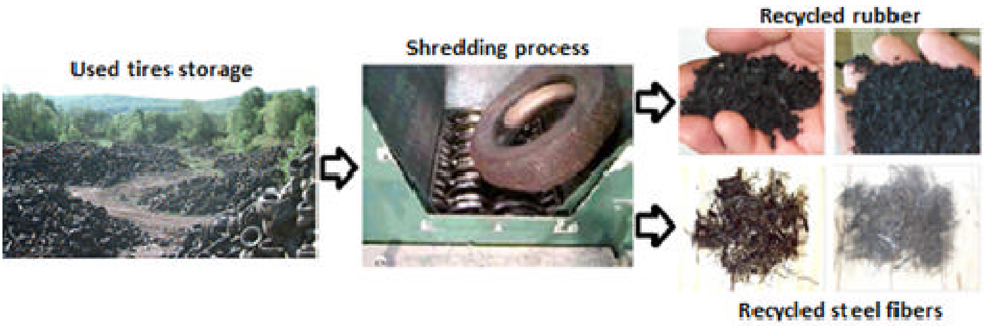

Figure 6. The recycling process by shredding of used tires

To obtain a more complete separation of components, the recycling of the post-consumed tires should be done in more than one stage.

3. CONCLUSIONS

The use of recycled materials in road practices is a new and efficient technology contributing to the sustainable development leading to both: the preserving of natural resources and the use of waste materials such as demolished cement concrete and steel fibers obtained by recycling of used tires.

Although from economical point of view the use of natural aggregates it is justified in some cases, a full analysis including preserving of natural resources and costs for the improvement of environmental parameters may justify the option of using the recycled materials.

Studies and researches actually undertaking in the frame of postdoctoral research program aim to develop new design principles for rigid pavements made with recycled aggregates.

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Nondestructive evaluation of physico-mechanical parameters of glass-based plate materials

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Summary

Glass-based plate materials are commercially produced as building blocks for surface treatment of walls and floors. For the evaluation of their physical mechanical and chemical parameters are commonly used destructive test methods. This article analyzes the possibility of using non-destructive investigation of selected physical and mechanical parameters of sintered glass-based building elements, and in the manufacture of secondary raw materials used. As outputs are listed in the evaluation of the regression function describing the degree of dependence parameters investigated destructive way or other normative and non-destructive method.

It was found out that the use of ultrasonic pulse method for the evaluation of glass-based plate materials is possible, but to a limited scope or as a guidance only for assessing the product performance coming directly out of a production line. Furthermore, it is necessary to take into account the dimensions of samples on which analyses have been carried out and also the ranges in which the examined parameters have been determined.

KEYWORDS: non-destructive testing, strength, water absorbility, plate materials.

1. INTRODUCTION

Non-destructive testing is a cheaper and often even less time-consuming alternative to the methods that involve destruction of the test material. Moreover, in many situations they seem more appropriate than some classical methods based on, for instance, sampling from a structure or destructive testing. Non-invasive methods have been used in the construction industry for more than 60 years and they have repeatedly confirmed their worth during that time. Today, there is a variety of methods which can be used to detect various characteristics of materials. In the construction practice, the most attention is classically paid to detecting the strength characteristics of materials used in structures.

One of the best-known non-destructive methods is, apart from others, an ultrasonic impulse method. This method is based on the principle of mechanical wave propagation in materials. It is used mainly where we want to find out the velocity of ultrasonic wave propagation, the quality of the tested environment, such as product uniformity, or some of its physico-mechanical parameters. The principle of the ultrasonic method is that repeated electrical pulses generate, in an exciter, tight bundles of damped mechanical oscillation. These mechanical pulses are then led into a test specimen and, after passing measured distance L, they are recorded by a sensor. The transit time of the ultrasonic waves is measured, i.e. time t from the pulse generation by the exciter until its impact is recorded by the sensor.

2. GLASS-BASED PLATE MATERIALS

These materials have seen a relatively great expansion in the last few years. Simply put, it is granular sintered glass with possible addition of modifying components. These components are, for instance, powdered pigments to achieve a desired hue. After trimming to a desired dimensional form and potential surface fineness correction, glass-based plate materials are used primarily for wall and floor tiling and building floors, both exterior and interior. Their excellent characteristics (e.g. high frost resistance, resistance to chemical action, high bending strength, colour stability, environmental friendliness) are a predisposition for the purpose of their use.

3. USE OF ULTRASONIC METHOD TO EVALUATE PARAMETERS OF AMORPHOUS MATERIALS

Given that the ultrasonic method is widely used mainly for evaluating the parameters of concrete, which is a crystalline material, within the framework of [4] researches were made on the use of ultrasonic methods for evaluating the parameters of amorphous materials, namely glass. Here it was found that many authors have already dealt and are still dealing with this rather complex and comprehensive issue. Their results are mostly positive in nature, when mainly various modifications of the ultrasonic method led them to the fact that there are some dependencies between the parameters studied non-destructively and by means of standard methods. The objectives of their studies have been, for instance, elastic modulus, density, temperature effect or the distribution of residual surface tension.

The following figure shows the typical texture of plate material based on granular heat-treated glass, which is a common commercial product. The figure shows that this material has more like a crystalline structure but, as already made determinations (e.g. fracture surface when determining bending strength) as well as theoretical work reveal, it is quite evident that they are amorphous materials with the potential presence of crystalline phase, but to a limited scope only.

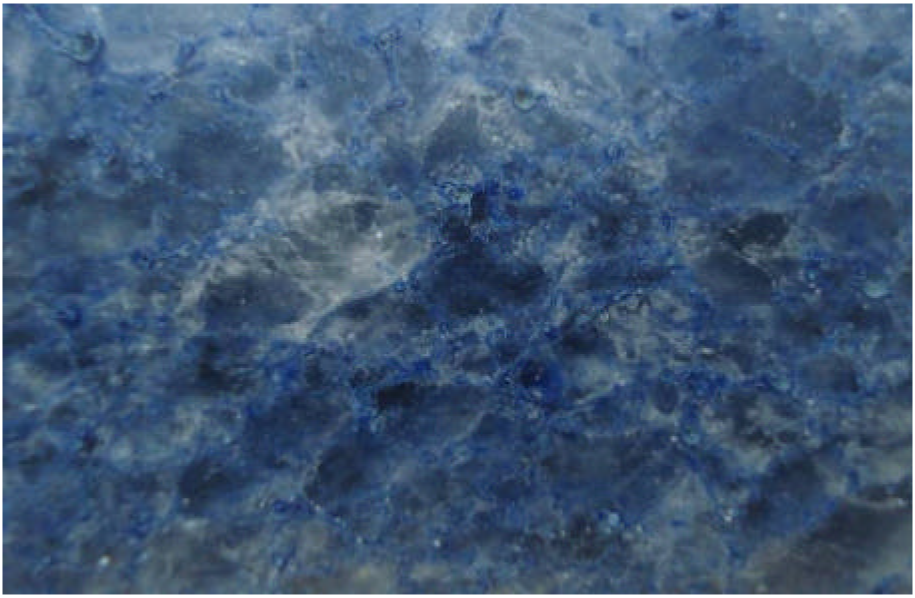

Figure 1. Characteristic texture (structure) of glass-based plate material – structure coloured with blue pigment

Given the nature of these materials, it appears to be very beneficial to use the ultrasonic pulse method for approximate evaluation of their selected parameters at the end of a production line without the need to use conventional destructive methods. It would, however, be necessary to modify this method, i.e. to select an appropriate probe type (exciters and receivers) etc.

4. EXPERIMENTAL PART

For the purposes of the research presented in this paper, test specimens of glass-based plate materials made from recycled glass coming from dismantled TV screens were used. Specifically, they were a combination of screens and cones of the fraction of 0 to 8 mm. Detailed specifications of recipes produced for experimental purposes are listed Table 1. Recipe marking contains three basic data. First, there is the raw material type – it is CRT (Cathode Ray Tube) in this case. Then the figures specify the used fraction of alternative fillers (e.g. 14 indicates fraction of 1 - 4 mm). Lastly, there is the maximum temperature in the area of maximum isothermal endurance.

Table 1. Results of determining ultrasonic velocity, absorbility and bending strength

Batch	Sample	Velocity [km.s^{-1}]	Water absorbility [%]	Bending strength [N.mm^{-2}]
CRT 01 700	1	4.80	0.21	18.0
	2	4.81	0.28	25.7
	3	4.70	0.10	25.5
	4	4.74	0.04	21.1
	5	4.50	0.14	19.1
CRT 08 800	6	4.71	0.14	31.9
	7	4.58	0.21	25.4
	8	4.65	0.13	27.1
	9	4.79	0.06	29.0
	10	4.75	0.14	27.6
CRT 48 700	11	3.35	2.30	5.4
	12	3.70	1.43	4.7
	13	3.64	1.95	3.4
	14	3.35	2.27	3.9
	15	3.44	2.65	4.9
CRT 08 960	16	4.86	0.22	40.6
	17	4.71	0.21	23.5
	18	4.35	0.11	32.6
	19	4.54	0.15	31.5
	20	4.25	0.20	33.0
CRT 48 800	21	4.69	0.17	18.8
	22	4.56	0.12	21.9
	23	4.77	0.18	19.6
	24	4.64	0.30	22.4
	25	4.94	0.21	18.3
CRT 18 800	26	5.14	0.14	28.7
	27	4.68	0.25	29.2
	28	4.66	0.17	36.9
	29	4.35	0.22	31.3
	30	5.03	0.05	32.4
CRT 14 800	31	4.48	0.11	27.9
	32	4.64	0.31	21.6
	33	4.91	0.12	24.5
	34	4.50	0.16	25.7
	35	4.61	0.20	23.5

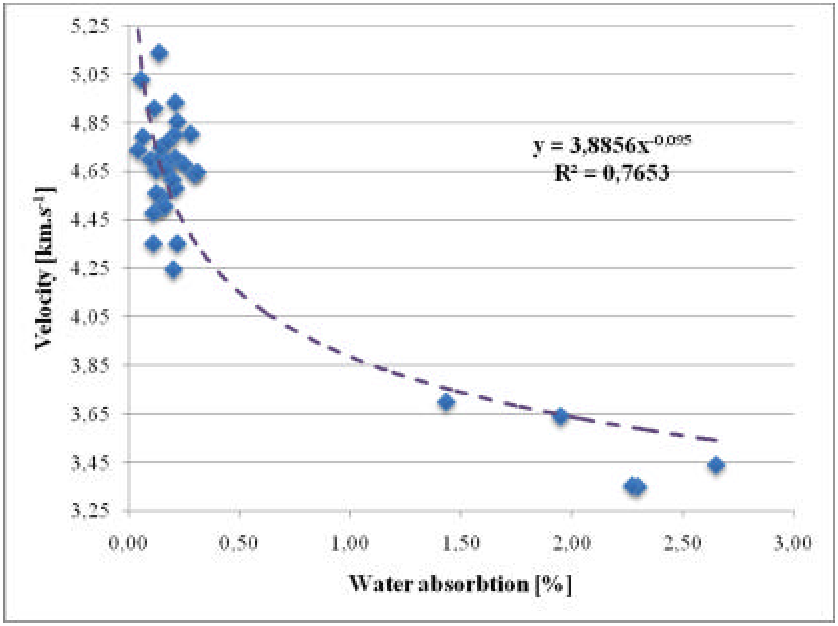

Figure 1. Graph on the dependence of ultrasonic velocity on water absorbility of test specimens – power function

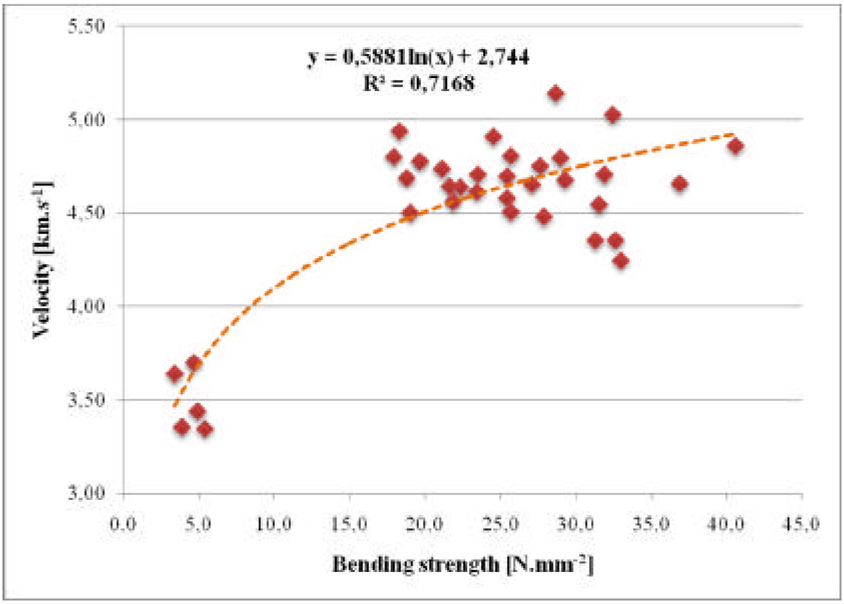

Figure 2. Graph on the dependence of ultrasonic velocity on bending strength of test specimens – logarithmic function

The graphical and tabular evaluations clearly show that the CRT 48 700 recipe has fundamentally different properties in relation to the others. This is mainly due to the absence of fine and medium fractions as well as low temperatures in the area of maximum isothermal endurance. Graphics were prepared for the dependence of ultrasonic velocity on selected characteristics of manufactured specimens, i.e. water absorption and bending strength. For each characteristic an equation was derived expressing the dependence of the relevant parameters. In the case of absorbility, power function was found as the most appropriate curve; this function takes the following form after putting relevant symbols:

$$E = \left(\frac{v}{3{,}8856} \right)^{\frac{1}{0{,}095}} \tag{1}$$

with the interval of this equation applicability being found for $v \in \langle 3{,}35;5{,}14 \rangle$, $E \in \langle 0{,}04;2{,}65 \rangle$ and the correlation coefficient is *R=0.8748*. However, considering the variability of the CRT 48 700 recipe test specimens, it is appropriate to limit the applicability interval of equation (1) to $E \in \langle 0{,}04;0{,}31 \rangle$.

The dependence of ultrasonic velocity on bending strength of glass-based plate materials is then given by a logarithmic equation in the following form:

$$R = e^{\left(\frac{v - 2{,}744}{0{,}5881} \right)} \tag{2}$$

with the interval of this equation applicability being found for $v \in \langle 3{,}35;5{,}14 \rangle$, $R \in \langle 3{,}43;40{,}59 \rangle$ and the correlation coefficient is *R=0.8466*. However, considering the variability of the CRT 48 700 recipe test specimens, it is appropriate to limit the applicability interval of equation (2) to $R \in \langle 17{,}95;40{,}59 \rangle$.

5. CONCLUSIONS

The results of the research presented in this paper point to the advantage of evaluating selected parameters of screen-glass-based plate materials using non-destructive glass. Specifically, the ultrasonic impulse method was used to evaluate or create a dependency between characteristics specified by standards and parameters specified non-destructively. Prediction equations were derived for absorbility and bending strength, with a relatively high degree of dependence of the observed variables was obtained within the relationships investigated. However, they should only be considered as a kind of basic or tentative with regard to the variance of the measured values and yet a small sample of the analyzed data. Therefore, the foregoing facts make it obvious that follow-up research is needed to refine the dependencies obtained, to take into account other essential criteria (e.g. chemical composition of materials investigated), the derivation of relations for other relevant parameters of glass-based plate materials (e.g. impact strength, resistance to sudden temperature changes) etc.

Acknowledgements

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Appendix

In this article the possibility of ultrasonic impulse method usage for physical and basic mechanical properties of plate materials based on glass is analyzed. Given that it is still not much explored area the research presented in the paper is mainly in primary phase. The obtained results show certain possibility of using of ultrasonic impulse method for evaluation some physical and mechanical properties of the investigated materials.

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Evaluation of the bearing capacity for the foundation soil in seismic condition

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Summary:

The paper main objective is to point out and compare the values of a soil bearing capacity, determined in seismic conditions, according to Eurocode 8 and GP 014-97 normative, for direct foundations.

The purpose of this evaluation is to quantify the degree to accomplish the design conditions regarding the resistance and safety in exploitation. For values already known of the loadings and physical-mechanical characteristics for a foundation soil have been determined the values for the bearing capacity of a strip footing with on imposed B width, according to the previous of the two norms.

KEYWORDS: strip footings, bearing capacity, partial coefficients, seismic conditions.

1. INTRODUCTION

The geotechnical design according to the rules and principles of normative Eurocode 8, implies acquiring and mastering of new concepts regarding the design statement approach. For spread foundations Eurocode 8 (informative annex F) proposes a method to calculate the bearing capacity in seismic conditions for cohesive and cohesionless soils. For the evaluation of bearing capacity is being used an equation that creates a link between soil resistance, seismic action effects and inertial forces in the soil.

2. CALCULATION OF SOIL BEARING CAPACITY IN LIMIT STATE UNDER SEISMIC CONDITIONS

For o strip foundation, solicited by a vertical permanent loading N_{Gk} = *100 kN/m* and a vertical variable loading N_{Qk} = *15 kN/m*, situated on the cohesive soil at depth *D = 1,1 m*, which is characterized by a silty clay with the natural unit weight *? = 17,8 kN/m*^{3}, cohesion *c' = 21 kN/m*^{2} and the internal friction angle *f = 18°*, will be determined, under seismic conditions, according to normative Eurocode 8 and GP 014-97, the bearing capacity values corresponding to a foundation base depth *B = 3 m.*

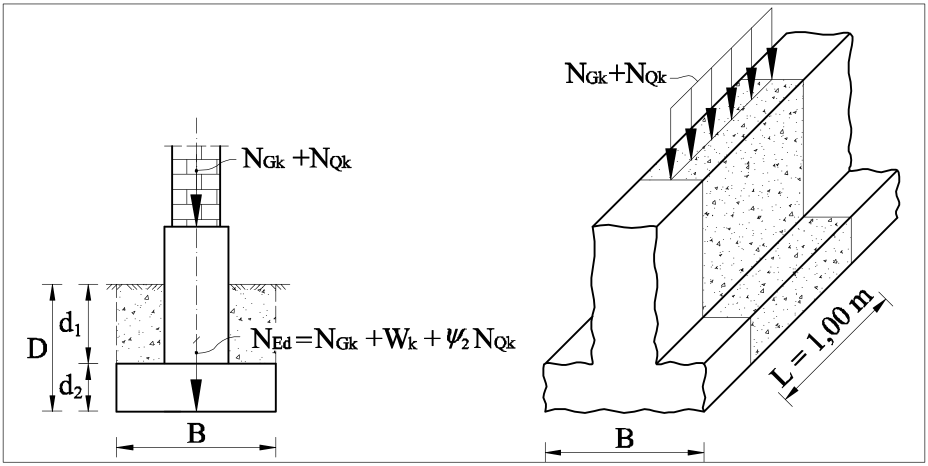

Fig. 1 Strip foundation – loadings and dimensions

2.1. Bearing capacity calculation according to SR EN 1998-5:2004 (Eurocode 8)

In order to evaluate the bearing capacity for spread foundations in seismic conditions according to Eurocode 8, part 5, must be verified fulfillment of general relation (F.1), presented in Annex F:

$$\frac{\left(1-e\overline{F}\right)^{cT}\left(b\overline{V}\right)^{cT}}{\left(\overline{N}\right)^{a}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{b}}+\frac{\left(1-f\overline{F}\right)^{c'M}\left(g\overline{M}\right)^{cM}}{\left(\overline{N}\right)^{c}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{d}}-1\le 0 \tag{1}$$

where,

$$\overline{N}=\frac{g_{Rd}N_{Ed}}{N_{max}},\quad \overline{V}=\frac{g_{Rd}V_{Ed}}{N_{max}},\quad \overline{M}=\frac{g_{Rd}M_{Ed}}{BN_{max}} \tag{2}$$

a, b, c, d, e, f, m, k, k', c_T, c_M, c'_M, *ß, ?*– numerical parameters depending on the soil type;

$?_{Rd}$ – partial coefficient of the model,

N_{Ed} – calculation value of normal stress on the horizontal foundation base,

V_{Ed} – calculation value of horizontal strength force,

M_{Ed} – calculation value of moment,

N_{max} – ultimate bearing capacity of the foundation loaded with a centric vertical force,

$$N_{max} = (p + 2)\frac{\bar{c}}{g_M} B \tag{3}$$

$\bar{c}$ - undrained shear strength of soil,
c_u - cohesion force for cohesive soils,
$\bar{F}$ - inertial force of soil, adimensional,

$$\bar{F} = \frac{r \cdot a_g \cdot S \cdot B}{\bar{c}} \tag{4}$$

? – soil gravity,
a_g – calculation values of soil acceleration in class *A*, (a_g = $?_I . a_{gR}$);
$?_I$ – importance factor with values from National Annex *SR EN 1998-1:2004/NA:2008*;
a_{gR} – peak reference value of soil acceleration in class *A;*
S – parameter characteristic to the soil class.

According to the national annex *SR EN 1998-1:2004/NA:2008*, the soil types classification (*A*, *B* ,*C*, *D*, *E*, *S1* and *S2*) is described in *SR EN 1998-1:2004.* This classification isn't applicable currently in Romania, being replaced with three site groups (Z_1, Z_2, Z_3).

The value of characteristic parameter will be *S = 1* for all the groups, and for the example, from the map with the classification of peak reference value of soil acceleration, results $a_{gR} = 0{,}20g$.

The general expression has the following boundaries:

$$0 < \bar{N} \le 1 , \quad \left|\bar{V}\right| \le 1 \tag{5}$$

Calculation values of seismic action effects N_{Ed}, V_{Ed}, M_{Ed}, result from the seismic grouping, defined according to norm SR EN 1990:2004:

$$SG_{k,j} \text{"+"} A_{Ed} \text{"+"} Sy_{2,i} Q_{k,i} \tag{6}$$

Calculation values for undrained cohesion (c_{ud}) and internal friction angle (f_d) can be determined by dividing the effective value to partial corresponding coefficients:

$$c_{ud} = \frac{c_u}{g_{cu}} \quad ; \quad j'_d = tan^{-1}\left(\frac{tan\, j'}{g_{j'}}\right) \tag{7}$$

Calculation of bearing capacity value and its evaluation, according to normative EC8, part 8, for strip foundation with width *B =3 m* are presented in table 1:

Table 1. Bearing capacity calculation according to Eurocode 8, part 5

Seismic grouping: $G_{k,i}"+"A_{Ed}"+"Sy_{2,i}Q_{k,i}$ - conf SR EN 1990:2004		
$y_2 = 0{,}4$; $g_I = 1{,}00$	$N_{Ed} = N_{Gk} + W_k + y_2 \cdot N_{Qk} = 174\ kN/m$	
$N_{Gk} = 100\ kN/m$; $N_{Qk} = 15\ kN/m$	$V_{Ed} = A_{Ed} = g_I \cdot A_{Ek} = g_I \cdot F_b = 20\ kN/m$	
$W_k = 68\ kN/m$ - footing weight	$M_{Ed} = V_{Ed} \cdot D = 22\ kNm/m$	
$g_{cu} = 1{,}4$	$c_u = 21\ kN/m^2$	$c_{ud} = c_u / g_{cu} = 15\ kN/m^2$
$g_{j'} = 1{,}25$	$j' = 18°$	$j'_d = tan^{-1}(tan\, j' / g_{j'}) = 14{,}57°$
$a = 0{,}70$; $b = 1{,}29$; $c = 2{,}14$; $d = 1{,}81$; $e = 0{,}21$; $f = 0{,}44$; $m = 0{,}21$ $k = 1{,}22$; $k' = 1{,}00$; $c_T = 2{,}00$; $c_M = 2{,}00$; $c'_M = 0{,}21$; $b = 2{,}75$; $g = 1{,}85$		
$N_{max} = 231{,}37\ kN/m$		
$r = 1{,}835\ t/m^3$; $S = 1$ $a_{gR} = 0{,}20g$; $a_g = g_I \cdot a_{gR} = 1{,}96$	$\overline{N} = 0{,}752$; $\overline{V} = 0{,}086$; $\overline{M} = 0{,}031$; $\overline{F} = 0{,}712$	
$\frac{\left(1-e\overline{F}\right)^{cT}\left(b\overline{V}\right)^{cT}}{\left(\overline{N}\right)^{a}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{b}} + \frac{\left(1-f\overline{F}\right)^{c'M}\left(g\overline{M}\right)^{cM}}{\left(\overline{N}\right)^{c}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{d}} - 1 = -0{,}0004 \le 0$		

2.2. Calculation of bearing capacity according to GP 014-97

The shallow foundation calculation in seismic condition according to normative GP 014-97, is based on calculation results in static condition (STAS 3300/2-85). The bearing capacity under to seismic action ($p_{cr,s}$) is represented by the critical pressure (p_{cr}) which is affected by an reduction coefficient ?:

$$p_{cr,s} = x \cdot p_{cr} \tag{8}$$

Reduction coefficient can be calculated with the relation:

$$x = \frac{1-h_s}{1-h_0} \tag{9}$$

where, for cohesive soil:

$$h_0 = \frac{s_v - s_h}{s_v + s_h + 2c \cdot ctg\, j} \cdot \frac{1}{sin\, j} \; ; \; h_s = \frac{\sqrt{4t_s^2 + s_v^2\left(1-K_0\right)^2}}{s_v\left(1+K_0\right) + 2c \cdot ctg\, j} \cdot \frac{1}{sin\, j} \tag{10}$$

K_0 – the lateral earth coefficient at rest,

s_v, s_h – the total unitary vertical stress and horizontal stress in soil at depth z:

$$z = D_f + \left[B/2 \cdot tg\left(45 + j'/2\right)\right]/2 \tag{11}$$

t $_s$ - the medium unitary tangential stress induced by the seismic action in soil:

$$t_s = 0{,}65 \cdot e \cdot K_s \cdot s_v \cdot r_d \tag{12}$$

e – coefficient based on foundation type; *e =1* for spread foundation
K_s – seismic coefficient:

$$K_s = a_{gR} / g \tag{13}$$

The analyzed site is situated in City Iasi, where according to *SR EN 1998-1:2004/NA:2008*, the peak acceleration value of the soil is:

$$a_{gR} = 0{,}20g \tag{14}$$

r_d – reduction coefficient depends on the depth z:

$$r_d = 1 - 0{,}015z, \tag{15}$$

Similarly to the critical pressure in static calculation (p_{cr}), the critical pressure in seismic calculation ($p_{cr,s}$) must be higher than the effective pressure:

$$p'_{ef} \leq m \cdot p_{cr,s} \tag{16}$$

The calculation values of the actions come from the group for seismic design, which is defined according to normative SR EN 1990:2004 (relation 6).

For the shear strengths parameters of soil, from the effective values will be determined the calculation values according to STAS 3300/1-85, for an assurance level a = 0,95:

- undrained cohesion $c = 21\ kN/m^2 \rightarrow c^* = 14\ kN/m^2$ (17)
- internal friction angle $f = 18° \rightarrow f^* = 9°$ (18)

Calculation of the bearing capacity under seismic conditions, according to GP 014-97, for strip foundation with width *B = 3 m*, is presented in table 2:

Table 2. Bearing capacity calculation ($p_{cr,s}$) according to GP 014-97

Seismic grouping: $G_{k,i}"+"A_{Ed}"+"Sy_{2,i}Q_{k,i}$ - according to SR EN 1990:2004	
$y_2 = 0{,}4$; $g_I = 1{,}00$	$N_{Ed} = N_{Gk} + W_k + y_2 \cdot N_{Qk} = 174\ kN/m$
$N_{Gk} = 100\ kN/m$; $N_{Qk} = 15\ kN/m$	$V_{Ed} = A_{Ed} = g_I \cdot A_{Ek} = g_I \cdot F_b = 20\ kN/m$
$W_k = 68\ kN/m$ - footing weight	$M_{Ed} = V_{Ed} \cdot D = 22\ kNm/m$
$c_u = 21\ kN/m^2$? $c_u^* = 14\ kN/m^2$	$j = 18°$? $j^* = 9°$
$p_{cr} = 183{,}53\ kN/m^2$; $0{,}9 \cdot p_{cr} = 165{,}18\ kN/m^2$	
$z = 1{,}978\ m$	$K_s = a_{gR}/g = 0{,}20$; $K_0 = 0{,}844$

$r_d = 0{,}97$ $e = 1$	$s_v = 35{,}21\ kN/m^2$; $s_h = 29{,}70\ kN/m^2$ $t_s = 4{,}44\ kN/m^2$ $h_0 = 0{,}146$; $h_s = 0{,}276$; $x = 0{,}847$
$p_{cr,s} = x \cdot p_{cr} = 155{,}44\ kN/m^2$; $0{,}9 \cdot p_{cr,s} = 139{,}89\ kN/m^2$; $p_{ef} = 63{,}35\ kN/m^2$	

The minimum width of the strip foundation, calculated according to GP 014-97, which minimal value complies with the condition $p'_{ef} = 0{,}9 \cdot p_{cr,s}$, is $B = 1{,}34\ m$.

3. RESULTS COMPARISON

The shallow foundations calculation in seismic conditions according to normative Eurocode 8, part 5 (annex F) and GP 014-97, present differences regarding the determination of actions calculation, soil shear strength parameters and the equations used for bearing capacity determination and verification.

In the example, for the same loading conditions, by using the verification equations of bearing capacity, results that in the strip foundation calculation according to Eurocode 8, for a width of the foundation base $B = 3\ m$, the bearing capacity is used completely by the calculation according to GP 014-97 the bearing capacity is being used only 45,3%.

Approximately the same perceptual difference is obtained if is considered the ratio between the foundation width, corresponding to the two normative:

$$\frac{B = 1{,}34m^{(GP014)}}{B = 3{,}00m^{(EC8)}} = 44{,}66\% \tag{19}$$

4. CONCLUSION

Based on the obtained values, we can consider that the strip foundations design using Eurocode 8, part 5 is more restrictive from the point of view of bearing capacity regarding the value obtained with GP 014-97. These results vary according to the design situation complexity.

Must be mentioned that in order to apply Eurocode 8 in the calculation methodology aren’t mentioned the coefficients used in the determination of calculation values for actions, values which are mentioned in EC 7.

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Evaluation of modulus of elasticity of concrete in terms of used entering components

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Summary

Elasticity modulus is one of the three basic parameters of static design of concrete structures. Determination of guiding values of strength and deformation characteristics is necessary for demanding building structures like bridges or high-rise buildings during designing stage. At this stage, it is necessary to determine specific requirements of concrete technology with respect to mix-design followed by testing of mix-design including determination of rheological properties of fresh concrete and tests of hardened concrete. Most frequently, compressive strength of concrete is determined to class the material into one of strength classes and static and/or dynamic elasticity modules are determined.

The paper describes evaluation of values of static elasticity modules including strength characteristics of concrete of different age. Several mix-designs were manufactured in laboratory conditions using cement of various strength classes, and mined and crushed aggregate of various origins. Test specimens were used for determination of compressive strength and static elasticity modulus. Tests were carried out in accordance with Standardized procedures. Finally, values of static elasticity modules were compared to guiding values stated in the Standard, which are inferred from compressive strength values. The results confirm a hypothesis that values of elasticity modulus cannot be inferred only from known strength characteristics, without further practical testing. This is important in particular for statically demanding structures, since character of individual input materials for designing concrete is considerably different compared to past years.

KEYWORDS: components of concrete, mix-designs, strength of concrete, elasticity modulus of concrete.

1. INTRODUCTION

Development of technology of concrete brings many changes of relationships among basic physico-mechanical parameters of hardened concrete. This trend should be eventually reflected in re-evaluation of dependencies ruling static design of concrete structures. In previous times, compressive strength as a basic characteristic of concrete was a sufficient datum form which other calculated characteristics were empirically calculated, like tensile bending strength, cross tension and static elasticity modulus. As a result of recent changes and new trends in the field of designing and application of concrete structures, durability and defined deformation properties of concrete have become complementary and sometimes even dominant requirements. In general, material with higher elasticity modulus shows less deformation. Measure of deformation depends on measure of stress, deformation properties of aggregate and cement stone, density, humidity and age of concrete. Values of material properties necessary for calculation of instantaneous and long-term deformation of concrete depend not only on strength class of concrete, but mainly on properties of aggregate and other parameters connected with mix composition. [1]

1.1. Mix-design and influence of input material on elasticity modulus of concrete

Concrete as composite material with different content and type of coarse aggregate or different mix-design shows various elasticity modulus [2]. Elasticity modulus of concrete is influenced by quality and proportions of individual components. Natural aggregate always shows higher value of elasticity modulus than hardened cement paste. Elasticity modulus of concrete lies between elasticity modulus of aggregate and elasticity modulus of cement stone [3], see Figure 1. Elasticity modulus of hardened cement paste is influenced by the same factors and compressive strength. Porosity has strong influence on elasticity modulus of cement stone. The higher porosity, the lower the value of elasticity modulus. Type of aggregate can also considerably influence elasticity modulus. It is not only kind of aggregate but also the quarry where the stone was mined, which can have strong impact on elasticity modulus of concrete. [2,4] The boundary between hardened cement paste and grains of aggregate can also influence elasticity modulus [3]; it is so-called transition zone.

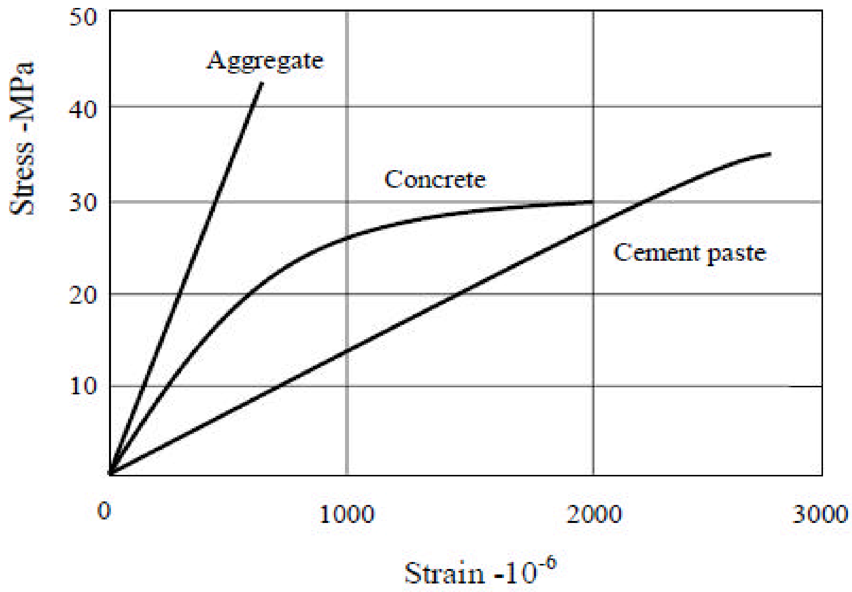

Figure 1. Relationship of stress and relative deformation for cement paste, aggregate and concrete [A.M.Neville, 1996][2]

Figure 1. shows elasticity modulus of concrete, which is different from elasticity modulus of hardened cement paste and aggregate. Elasticity modulus of cement paste lies between 5 and 25 GPa, elasticity modulus of natural stone from 30 to 100 GPa [3] depending on mineralogical composition.

1.2. Empirical formulation of elasticity modulus

Elasticity modulus of common concrete is related to the value of compressive strength. CSN EN 206-1 [5] states following empirical formula for calculation of elasticity modulus of concrete:

$$E = 22\,(f_c/10)^{0,3}$$

Relationship or dependency between elasticity modulus of concrete and its compressive strength is also defined in CSN EN 1992-1-1 EuroCode 2: Designing of concrete structures – Part 1-1: General regulations and regulations for civil constructions [6]. Hence, elasticity modulus is empirically related to strength class. Percental formulation of decrease of stated values with respect to different kind of aggregate is also stated. CSN EN 1992-1-1 takes into account only material substance of used aggregate, not the kind and amount of additives and admixtures, w/c ratio, type of transport and placing etc. [7]

Table 1. Basic strength characteristics and elasticity modulus of concrete according to CSN EN 1992-1-1 [6]

Class of concrete	Characteristic cylinder strength fc (MPa)	Characteristic cube strength fc, cube (MPa)	Mean value of compressive strength fcm (MPa)	Elasticity modulus Ecm (GPa)
C 12/15	12	15	20	27
C 16/20	16	20	24	29
C 20/25	20	25	28	30
C25/30	25	30	33	31
C 30/37	30	37	38	32
C 35/45	35	45	43	34
C 40/50	40	50	48	35
C 45/55	45	55	53	36
C 50/60	50	60	58	37
C 55/67	55	67	63	38
C 60/75	60	75	68	39
C 70/85	70	85	78	41
C 80/95	80	95	88	42
C 90/105	90	105	98	44

These empirical relationships cannot grasp the situation of modern concrete, since coarse aggregate in modern concrete is very important with its strong influence on mechanical properties [4]. However, calculated values of elasticity modules do not always reach real values of individual compressive strength values [3].

2. EXPERIMENTAL VERIFICATION

2.1. Design of composition of concrete

Experimental part focused in particular on comparison of result values of static elasticity modules. Different mix-designs were proportioned with respect to type of used aggregate. Mined and crushed aggregate originated from different quarries (‘”A” and “B”) and aggregate was aslo of different size fractions. Mix-designs used different types of cement. i.e. classic Portland cement and Portland slag cement. The cements were of classes CEM I 42,5 R and CEM II/A-S 42,5 N. All mixes contained water and identical type of superplasticizer made by Stachema. Dosage of individual components and mix-design is given in Tables 2 and 3.

Table 2. Mix-designs for concrete with crushed aggregate of different size fractions from different quarries

	Aggregate type "A"		Aggregate type "B"	
Mix	A1	A2	B1	B2
CEM I 42,5 R	-	-	410	440
CEM II/A-S 42,5 N	430	410	-	-
Aggregate 0-4	834	859	750	725
Aggregate 4-8	-	-	220	210
Aggregate 8-16	919	909	860	850
Superplasticizer	2,8	2,6	3,3	3,7
Water	159	160	151	154
water-cement ratio	0,37	0,39	0,37	0,35

Table 3. Mix-designs for concrete with mined aggregate from different quarries

	Aggregate type "A	Aggregate type "B"
Mix	A1	B1
CEM I 42,5 R	-	380
CEM II/A-S 42,5 N	380	-
Aggregate 0-4	740	750
Aggregate 4-8	210	220
Aggregate 8-16	430	440
Aggregate 16-22	480	480
Superplasticizer	2,4	3,0
Water	149	138
water-cement ratio	0,39	0,36

2.2. Strength and elasticity-deformation characteristics

Test specimens were made from individual mixes to determine properties of hardened concrete. Testing cubes of side 150 mm were made for compressive strength test and testing beams of dimensions 100 x 100 x 400 mm were made for static elasticity modulus test. Test specimens were placed in laboratory conditions until the time of testing. Tests were carried out after required periods, i.e. 3, 7 and 28 days for concrete with "A" type of aggregate and only after 28 days form concrete with "B" type aggregate.

Table 4. Compressive strength and static elasticity modulus values of concrete with crushed stone of different size fractions and from different sites

	Aggregate type “A”		Aggregate type “B”	
Mix	A1	A2	B1	B2
fc 3 days	26,3	28,3	-	-
fc 7 days	38,6	39,6	-	-
fc 28 days	44,4	53,4	70,5	73,5
Ec 3 days	26500	25500	2800	25000
Ec 7 days	31000	30500	29500	28000
Ec 28 days	31500	32500	34500	32000

Table 5. Compressive strength and static elasticity modulus values of concrete with mined stone from different sites

	Aggregate type “A”	Aggregate type “B”
Mix	A1	B1
fc 3 days	39,6	-
fc 7 days	51,1	-
fc 28 days	61,9	62,9
Ec 3 days	31500	35000
Ec 7 days	34000	38500
Ec 28 days	38000	41500

Note:
- fc – compressive strength of concrete (MPa)
- Ec – static compression elasticity modulus of concrete (MPa)

2.3. Comparison of static elasticity modulus and guiding values stated by the Standard

Finally, values of static compressive elasticity modulus after 28 days were compared to guiding values stated in CSN EN 1992-1-1. For this reason, compressive strength values were ranged into individual strength classes of concrete. Consequently, matching values stated in the Table of static elasticity modulus were compared according to individual strength classes. The results of comparison or real and tabulated values of static elasticity modulus are given in Tables 6 and 7.

Table 6. Comparison of real and tabulated values of static elasticity modulus for concrete with crushed stone of different size fractions and from different sites

	Aggregate type “A”		Aggregate type “B”	
Characteristic	A1	A2	B2	B3
Real compressive strength	44,4	53,4	70,5	73,5
Strength class according to CSN EN 206-1 [6]	C 30/37	C 35/45	C 55/67	C 55/37
Real value of elasticity modulus (MPa)	31500	32500	34500	32000
Elasticity modulus according to CSN EN 1992-1-1 (MPa) [3]	32000	34000	38000	38000

Table 7. Comparison of real and tabulated values of static elasticity modulus for concrete with mined aggregate from different sites

	Aggregate type “A	Aggregate type “B”
Characteristic	A1	B1
Real compressive strength (MPa)	61,9	62,9
Strength class according to CSN EN 206-1 [6]	C 50/60	C 50/60
Real value of elasticity modulus (MPa)	38000	41500
Elasticity modulus according to CSN EN 1992-1-1 (MPa) [3]	37000	37000

3. CONCLUSIONS

The paper focused on research of elasticity modulus of concrete of different mix-designs. Individual mix-designs used various types of cement and various types origins of aggregate. Compressive strength of concrete with crushed stone was positively influenced by three-fraction composition of aggregate with pure Portland cement. Compressive strength of this type of concrete - labeled “B” - was higher by as much as 22 MPa after 28 days. This result could be observed also with concrete with mined aggregate, where both types of aggregate or both types of concrete had three-fraction composition of aggregate and values of compressive strength were identical. This can be unambiguously understood as disproof of an idea that such a difference of strength could be caused only by the type of cement. As for elasticity modulus of concrete with crushed aggregate, mix-designs with aggregate from both quarries reached almost identical values. However, values of concrete with crushed aggregate of type ”A” were higher and comparable to values of concrete with higher compressive strength. As for concrete with mined aggregate, mix-designs with type “B” aggregate showed higher increase of values of elasticity modulus. Development of the value from the age of 3 days to the age of 28 days was about 3500 MPa, unlike mix-designs with type “A” aggregate. Comparison of real and tabulated values of static elasticity modulus brought various and contradictory results. Concrete with crushed aggregate of type “A” showed comparable values, however, concrete with aggregate of type “B” did not reach tabulated values. The difference between elasticity modules is around 5 000 MPa. As for crushed stone, concrete with both types of aggregate reached sufficient values. In general, the thought of advantageous values of static elasticity modulus of concrete with mined aggregate can be accepted. As for concrete with crushed aggregate, it can here and there reach tabulated values, however only with carefully designed composition of aggregate. Nevertheless, mineralogy, site and quality of processing are very important in particular for crushed aggregate. Hence, the experiment proves that it is not always possible to rely on recommended tables referring to guiding values, which, as practice shows, can differ from real values of current concrete used on concrete structures.

ACKNOWLEDGEMENTS

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Main aspects on existing strategies in risk and emergency management for roads

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Summary

The risks, due to natural and man-made hazards, that can affect the roads are increasing. Recent studies indicate that there is an ascending trend of the fatalities on roads and highways. This paper is meant to be a short situation assessment and in the same time an introduction in the field of risk and emergency management for roads. After definition and classification on various risks affecting the road network, the main accomplishments undertaken in the frame of PIARC technical committee TC3 are presented, including the toolbox that have been developed. Finally the main strategies established at the national level are presented with comments and proposals for their implementation into road engineering process.

KEYWORDS: risk management, natural hazards, man-made hazards.

1. INTRODUCTION

1.1. Definition

Risk management is the identification, assessment and prioritization of risks (defined in ISO 31000 as the effect of uncertainty on objectives, whether positive or negative) followed by coordinated and economical application of resources to minimize, monitor, and control the probability and/or impact of unfortunate events or to maximize the realization of opportunities [1].

1.3. Types of risks

There are various ways in which risks can be classified. Their categorization is made by taking into account the aspects that may be involved: causes, impact, interests and so on. It's up to the organization to choose the proper classification in order to satisfy the needs of the internal risk management process.

The most common classification is by splitting the risks into two main categories: *natural risks* and *man-made risks*.

Table 1. Risks classification [2]

Natural risks	Man-made risks
- earthquakes, - floods, - landslides, - avalanches, - rock fall, - forest fires, - snow/ice/wind storm, - heavy rain, - fog, - drought, - volcanic eruption.	- traffic accidents, - work accidents, - transport of dangerous goods, - airplane/ship/train crash, - industrial accidents, - wartime explosives/mines, - strikes, - terrorism/ vandalism, - traffic congestion, - dam collapse, - epidemical, - gas explosion, - overloading(height, weight).

1.2. Risk management process scheme

Risk analysis is normally said to include risk identification and risk evaluation. Identification is a process which consists in scanning of the world for possible hazards pertinent to whatever interests might be involved. Evaluation is the more sophisticated step where the decision-making science or methodology should be use.

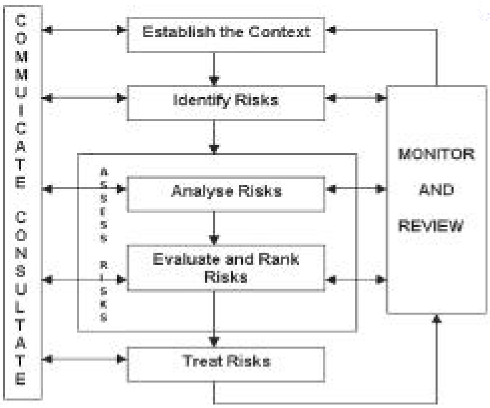

Figure 1. Risk management process [2].

2. PIARC ACHIEVEMENTS

2.1. PIARC general mission

The World Road Association (PIARC) is a non-political and non-profit making association. It was granted consultative status to the Economic and Social Council of United Nations in 1970 [3].

PIARC exists to serve all its members by:

- being a leading international forum for analysis and discussion of the full spectrum of transport issues, related to roads and road transport,
- identifying, developing and disseminating best practice and giving better access to international information,
- fully considering within its activities the needs of developing countries and countries in transition,
- developing and promoting efficient tools for decision making on matters related to roads and road transport [3].

2.1. Technical Committee TC 3.2 – goal, strategy, technical toolbox

The goal of the Strategic Theme 3 of PIARC is to improve the safe and efficient use of the road system, including the movement of people and goods on the road network, while effectively managing the risk associated with road transport operations and the natural environment.

The Technical Committee on Risk Management for Roads(TC 3.2), from PIARC, has been working to achieve the main goal, mentioned above, and to meet the ever increasing demand of risk management and to serve as a link between practice, sciences, policy making and decision making in the search for the risk management for roads. TC 3.2 lays special emphasis on integrated risk management with expanded research into risk assessment, decision making processes and security issues. There are three issues as shown in Table 2, and three working groups that are responsible for each issue have been established [2].

Table 2. Terms of reference for TC 3.2 [2]

Issue 1 - Introduce risk management techniques in the road sector	
Strategies	**Outputs**
- collect and analyze information about Integrated Risk Management from a strategic organizational standpoint, - collect information about the use of quantitative risk assessment/management tools and develop best practices/ lessons	- recommendations on how risk management can be used in an organization to guide programs/projects - report on existing practices, Model Integrated Risk Management Framework that can be used as a Guide,

learned on risk based decision making, - study how security risks/vulnerability can be used to assess major transportation alternatives and impact the decision making process.	- quantitative risk assessment toolbox of techniques and methodologies which can be applied to the transportation community.
Issue 2 - Introduce risk management for mega-projects	
Strategies	**Outputs**
Study the use of risk assessment tools on mega-projects and assess their success	Guidance on better use of risk management on mega- projects to maintain public trust and confidence
Issue 3 – Improve highway system security	
Strategies	**Outputs**
Investigate the application of risk management principles to the reduction of risk for the highway system	Vulnerability assessment model for critical transportation infrastructures

The goals and results of the three working groups:

Table 3. Working groups [2]

Goals		Results
Group 1	Introduce risk management techniques in the road sector	- risk management toolbox and techniques, - framework for integrated risk management, - principles and techniques common to all risk management applications - examples of the use of risk management from an organizational standpoint.
Group 2	Introduce risk management for mega-projects	- advise for better use of risk management on mega-projects - risk management toolbox, including the definition of mega-projects, risk assessment of natural and technological hazards.
Group 3	Improve highway systems security	- risk management toolbox, - vulnerability assessment model for critical transportation infrastructures.

Another important part of the TC 3.2 report is the technical toolbox which is a database of policy, techniques and operation(maintenance) technologies\tools with inspection facility for road management, which consists of inventory sheets and their appendix.

The inventory sheets are prepared to introduce the risk management technology used mainly in Japan and in New Zealand to the developing countries, and the risk management technology/tools from different countries will be added to them. The inventory sheets aim to assist budgeting and road management with easy application of risk management technologies/tools [4].

Advantages of the inventory sheets are:
- provision of general idea of technologies/tools, precedents, cost etc.,
- easy decision-making to adopt best technologies/tools for risk management process according to the summary in each sheet,
- easy revision and expansion based on their development in digital availability,
- usage as an effective tools of technology transfer to developing countries [4].

3. NATIONAL STRATEGIES

Romania, alike other countries in the world, is exposed to a wide range of hazards due to the interaction of natural factors with the demographic, social and anthropogenic ones and with the infrastructure elements such as buildings, roads, railways etc.

Following the developing and emerging countries, Romania, elaborated, based on different analysis and surveys, a series of strategies such as:
- National Strategy for communication and public information in case of emergency situations,
- National Strategy for risk management in case of floods,
- Romanian National Strategy concerning the climate changes,
- National strategy the management of road traffic safety,
- National strategy for the management of emergency situations on the public roads.

The road safety is linked with national policies for road development. So that the efforts of the authorities to be coordinated and to response in the same time, the risk management plans should be approached in an integrated manner. This is the aim of the national strategies.

The specific objective is to restrict on the long term the social, economical and environment costs of the climate change and hazards impact.

The main activities, in the national strategies, of the risk management are:
- preventive activities,
- operative management, during the hazards,
- post hazard measures.

The institutional structure is the National System of Management of Emergencies, composed of the government and organized as a network of organizations, bodies and skill in emergency management, set up by level or area of competence, which has the infrastructure and resources required to perform in the field. National System of Management of Emergencies, subject to Government Emergency Ordinance no. 21/2004, is composed of permanent activity structures and structures with temporary work.

4. CONCLUSIONS

On the foundation of the actual strategies, Romania, by using PIARC's experience and implementing their procedures and methodologies, can make big steps in the development of risk management by introducing management techniques in road sector and dissemination of the road risk management technology.

Analyzing the risks witch may occur during the entire process of a road network(Planning – Design – Construction – Operation - Reconstruction) will have a positive effect on the following aspects:

- economical development,
- relation between entrepreneurs and investors,
- efficiency of road network,
- inventory of the assets,
- current data reflecting the condition of the assets,
- land use,
- environment impact.

Knowing the effects and vulnerabilities generated by climate change and applying the measures stipulated in the national strategies will lead to significant improvements in the road sector by reducing the time and costs and also by reaching the same level of development with the other member states of European Union.

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Nonlinear modeling of liquefaction behavior of sand-silt mixtures in terms of strain energy

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Summary

New nonlinear correlations were developed to evaluate the liquefaction resistance of sandy soils using and a hybrid method coupling genetic programming (GP) and simulated annealing (SA), called GP/SA. The derived correlations relate the soil initial parameters to the strain energy density required to trigger liquefaction, denoted as capacity energy. To verify the applicability of the correlations, they were employed to estimate the capacity energy of parts of test results that were not included in the analyses. The external validation of the correlations was further verified using several statistical criteria recommended by researchers. A traditional GP analysis was performed to benchmark the GP/SA-based correlations against a classical approach. For more verification, the correlations were applied to some downhole array data of real earthquake case histories and some liquefaction centrifuge tests. The contributions of the parameters affecting the capacity energy were evaluated through a sensitivity analysis. A subsequent parametric analysis was carried out and the trends of the results were confirmed via some previous laboratory studies. The GP/SA correlations are capable of effectively capturing the liquefaction resistance of a number of sandy soils. The verification phases confirm the efficiency of the models for their general application to the assessment of the strain energy at the onset of liquefaction. The proposed correlations reach a significantly better prediction performance than the models found in the literature. The GP/SA correlations can reliably be used for pre-design purposes in that they were developed based on widely dispersed experimental results for various types of sandy soils. The proposed simple and straightforward prediction equations may be used as a quick check on solutions developed by more time consuming and in-depth deterministic analyses.

KEYWORDS: Liquefaction; Capacity energy; Genetic programming; Simulated annealing; Numerical modeling.

1. INTRODUCTION

Soil liquefaction is one of the most complex phenomena studied in geotechnical earthquake engineering. Liquefaction is the phenomenon of vanishing intergranular stresses as a material response to some loading paths. These loading paths can be isochoric paths. In practical cases, the isochoric situation plays an important role since it corresponds to an undrained loading. Liquefaction is commonly considered as a specific feature of loose and saturated sandy soils. The presence of water appears to be necessary only to allow easy verification of the isochoric condition [1]. The liquefaction phenomenon can be caused by seismic shaking, nonseismic vibration or waved-induced shear stresses. This phenomenon is a source of damage and destructive failures in various types of structures. The seriousness of potential failures of critical structures due to liquefaction led to massive research efforts to understand this phenomenon. Several procedures are developed to evaluate the liquefaction potential in the field. The available liquefaction evaluation procedures are categorized into three main groups: (1) stress-based procedures, (2) strain-based procedures, and (3) energy-based procedures [2]. The stress-based procedure is the most widely used liquefaction assessment method proposed by Seed and Idriss [3]. Dobry et al. [4] proposed the strain-based procedure as an alternative to the empirical stress-based procedure. This method was derived from the mechanics of two interacting idealized sand grains and then generalized for natural soil deposits. The basic elements of both the stress and strain methods are incorporated in the formulation of the energy-based method. In this method, the amount of total strain energy at the onset of liquefaction is obtained from laboratory testing or field recorded data. In a typical cyclic (triaxial or simple shear) laboratory test, the stress, strain and pore pressure time histories are recorded. Hysteresis loops can be generated from these stress and strain time histories. Figure 1 illustrates a typical hysteresis loop from a typical stress-controlled cyclic triaxial test. The strain energy for each cycle of loading is equivalent to the area inside the hysteresis loop. In other words, this area represents the dissipated energy per unit volume of the soil mass, denoted as dissipated energy density [2].

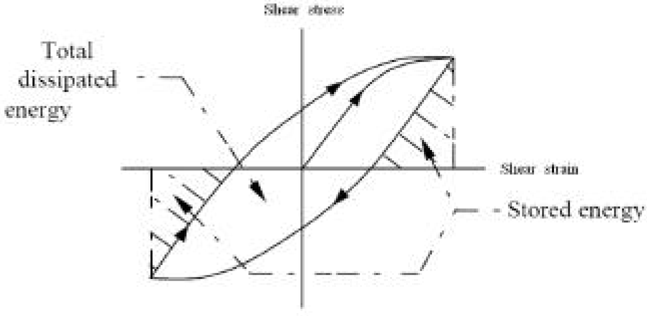

Figure 1. A typical hysteresis shear stress-strain loop [2]

This is based on the idea that during deformation of cohesionless soils under dynamic loads part of the energy is dissipated into the soil. The instantaneous energy and its summation over time intervals are computed until the onset of liquefaction. The summation of the energy at this time is used as the measures of the capacity of the soil sample against initial liquefaction occurrence in terms of the strain energy (capacity energy).The energy-based approach has several advantages in comparison with the other existing methods to evaluate the liquefaction potential of soils. Some of its most important advantages are: (i) energy is directly related to the intensity of dynamic loading; (ii) it is not necessary to decompose the shear stress history to find the equivalent number of uniform cycles for selected stress or strain levels; (iii) it is practically independent of the load waveform. Since there is no need to model a complicated random stress history in laboratory experiments, a simple sinusoidal pattern of loading can be used in all cases; and (iv) it is generally independent of the type of testing device which facilitates the comparison of results obtained in different laboratories [5]. However, the complexity of the liquefaction behavior suggests the necessity of developing more comprehensive models to assess it. Numerous studies have been carried out to propose energy-based models for the evaluation of the liquefaction potential. Several researchers have tried to explore a correlation between the potential of pore pressure build-up and the dissipated stress–strain energy in liquefiable soils, independent of the shear stress history [6]. Several researchers such as Simcock et al. [7] developed models to correlate the pore water pressure increment ratio and dissipated strain energy density, loading parameters, initial parameters of soils, initial effective confining pressure, and some calibration parameters. In recent years, Green et al. [8] developed an energy-based model on the basis of the stress-controlled cyclic triaxial test data on sand samples. Most of these relationships were derived by performing a multiple linear regression (MLR) analysis. Despite the considerable accuracy of such models, they were developed based on limited number of cyclic tests on sand. Also, in the conventional regression analyses, a linear relationship is often assumed between the outcome and the predictor variables, which is not always true. On the other side, only two groups of researchers, [9, 10], have taken into account the important role of the fines content in liquefaction behavior evaluation.

Several computer-aided data mining approaches have been developed by extending developments in computational software and hardware. Artificial neural networks (ANNs) are the most widely used pattern recognition procedures. They have rarely been applied to the assessment of the liquefaction resistance with emphasis on energy-based hypotheses. Baziar and Jafarian [9] developed an ANN-based model to establish a correlation between soils initial parameters and the strain energy required to trigger liquefaction in sands. Chen et al. [11] presented a seismic wave energy-based method with back-propagation neural networks to assess the liquefaction probability. Despite the acceptable performance of ANNs, they do not give a definite function to calculate the outcome using the input values.

Genetic algorithm (GA) is a powerful stochastic search and optimization method based on the principles of genetics and natural selection. GA has been shown to be suitably robust for a wide variety of complex geotechnical problems (e.g. [12]. Genetic programming (GP) [13] is an alternative approach for behavior modeling of geotechnical engineering tasks. GP may generally be defined as a specialization of GA where the solutions are computer programs rather than fixed-length binary strings. The main advantage of GP over the conventional statistical methods and other soft computing tools is its ability to generate prediction equations without assuming prior form of the existing relationship. In contrast with ANNs and GA, application of GP in the field of civil engineering is quite new and original. GP and its variants have recently been used to derive greatly simplified formulas for geotechnical engineering problems (e.g. [14]).

Simulated annealing (SA) is a general stochastic search algorithm used for solving optimization problems. The Metropolis algorithm, the foundation of SA, was proposed by Metropolis et al. [15] to simulate the annealing process. This algorithm was first applied to optimization problems by Kirkpatrick et al. [16] and Cerny [17]. SA is very useful for solving several types of optimization problems with nonlinear functions and multiple local optima. Folino et al. [18] combined GP and SA to make a hybrid algorithm with better efficiency. They used the SA acceptance strategy to select new individuals. It was shown that introducing this strategy into the GP process improve the simple GP profitably. Applications of the hybrid GP/SA technique to solve problems in civil engineering are conspicuous by their near absence. Recently, Alavi et al. [19] utilized this hybrid method to simulate the behavior of flow number of asphalt mixtures.

The main purpose of this paper is to utilize the GP/SA techniques to obtain generalized relationships between the energy per unit volume dissipated in generating liquefaction and the factors affecting it. A comparative study was conducted between the obtained results and those of the models found in the literature. A comprehensive database of previously published tests results was employed to develop the proposed correlations.

2. GENETIC PROGRAMMING

GP is a symbolic optimization technique that creates computer programs to solve a problem using the principle of Darwinian natural selection [13]. The breakthrough in GP then came in the late 1980s with the experiments on symbolic regression. GP was introduced by Koza [13] as an extension of genetic algorithms (GAs). Most of the genetic operators used in GA can also be implemented in GP with minor changes. The main difference between GP and GA is the representation of the solution. The GP solutions are computer programs that are represented as tree structures and expressed in a functional programming language (like LISP) [13]. GA creates a string of numbers that represent the solution. In other words, in GP, the evolving programs (individuals) are parse trees than can vary in length throughout the run rather than fixed-length binary strings. This classical GP approach is referred to as tree-based GP. A population member in tree-based GP is a hierarchically structured tree comprising functions and terminals. The functions and terminals are selected from a set of functions and a set of terminals. For example, function set F can contain the basic arithmetic operations (+, -, ×, /, etc.), Boolean logic functions (AND, OR, NOT, etc.), or any other mathematical functions. The terminal set T contains the arguments for the functions and can consist of numerical constants, logical constants, variables, etc. The functions and terminals are chosen at random and constructed together to form a computer model in a tree-like structure with a root point with branches extending from each function and ending in a terminal. An example of a simple tree representation of a GP model is illustrated in Figure 2.

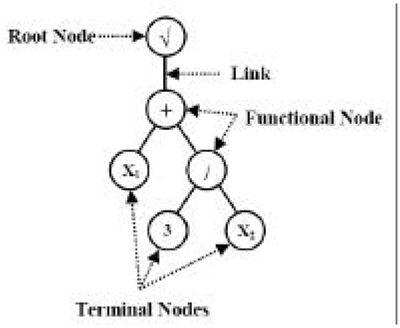

Figure 2. The tree representation of a GP model $v(X_1 + 3/X_2)$

The creation of the initial population is a blind random search for solutions in the large space of possible solutions. Once a population of models has been created at random, the tree-based GP algorithm evaluates the individuals, selects individuals for reproduction, generates new individuals by mutation, crossover, and direct reproduction [13]. During the crossover procedure, a point on a branch of each solution (program) is selected at random and the set of terminals and/or functions from each program are then swapped to create two new programs. The evolutionary process continues by evaluating the fitness of the new population and starting a new round of reproduction and crossover. During this process, the tree-based GP algorithm occasionally selects a function or terminal from a model at random and mutates it.

2.1. Hybrid genetic programming-simulated annealing algorithm

In this paper, a GP with a SA-based selection strategy is employed for developing the prediction correlations. In this coupled algorithm, the SA strategy is used to select new individuals [20, 21]. SA makes use of the Metropolis algorithm [15] for computer simulation of annealing. Annealing is a process in which a metal is heated to a high temperature and then is gradually cooled to relieve thermal stresses. During the cooling process, each atom takes a specific position in the crystalline structure of the metal. By changing the temperature this crystalline structure changes to a different configuration. An internal energy, E, can be measured and assigned to each state of crystalline structure of the metal which is achieved during the annealing process. At each temperature within the annealing process, if the temperature does not decrease quickly, the atoms are allowed to adjust to a stable equilibrium state of least energy. It is evident that changing of the crystalline structure of a metal, through the annealing, is associated with a changing of the internal energy as ?E. However, as the metal temperature drops down gradually, the overall trend of changing internal energy follows a decreasing process but sometimes the energy may increase by chance. The crystalline structure of a metal achieves near global minimum energy states during the process of annealing. This process is simulated by SA to find the minimum of a function in a certain design space. The objective function corresponds to the energy state and moving to any new set of design variables corresponds to a change of the crystalline structural state.

2.2.1. A coupled algorithm of GP and SA

Considering the above explanations for GP and SA, the coupled GP/SA algorithm uses the following main steps to evolve a computer program [20, 21]:

I. A single program is initially created at random. This is the “parent” program for the first repetition of the learning cycle.

II. The parent program is copied.

III. A search operator, crossover or mutation, transforms the copy of the parent program. The transformed copy is called “child” program or “offspring” program. The crossover operator produces two children programs. But only one of these programs is compared with the parent as a candidate to replace the parent program. Which of the two children is used depends on the value of the offspring choice parameter.

IV. The fitness value of the both parent and child program is calculated.

V. Based on the fitness value of the child and parent program, the SA algorithm decides whether to replace the parent program with the child program. If the child has better fitness than the parent, the child always replaces its parent. If the child has worse fitness than the parent, the child replaces the parent probabilistically. The probability of replacement depends on how much worse the fitness of the child is than the parent and also what is the SA temperature, T. As the annealing process continues, T is gradually reduced at each (at n) iteration. This means that, for the program, the probability of replacing a worse child to a better parent gets lower and lower as the run continues. If the child program replaces the parent program then the child program becomes the new parent for the next cycle. Alternatively, if the parent program is not replaced by the child, it remains as the parent program for the next cycle.

VI. If the termination or convergence conditions are satisfied, the process is terminated. Otherwise, the process is continued going step III.

3. DEVELOPMENT OF CORRELATIONS FOR ENERGY-BASED LIQUEFACTION ASSESSMENT

Soil liquefaction phenomenon involves progressive intergrain contact deformation, slip, reorganization of contacts, and eventual collapse of soil skeleton. The mechanical analysis of liquefaction shows that the volume variation rate imposed by the material flow rule (i.e. dilatancy angle) has to be lower than the volume variation rate imposed by the loading path. In this case, the effective stresses are decreasing possibly to zero. Thus, the dilatancy/contractancy angle plays a significant role in liquefaction process [1]. According to the experimental and theoretical studies, the dilatancy angle is a function mainly of the granular material, of its relative density and of the initial confining pressure [22]. Therefore, in a rational manner the main parameters which affect the liquefaction potential will be the grain size distribution of material, the initial relative density and the initial effective mean confining pressure. In this work, the GP/SA approach was employed to obtain meaningful relationships between the level of energy required for liquefaction of sands and the influencing parameters. The main objective is to provide alternative formulations to the conventional regression-based equations. The most important factors representing the liquefaction behavior were selected based on an extensive trial study and literature review. Consequently, the capacity energy formulation was considered to be as follows:

$$Log(W)= f\left(s'_{mean}, D_r, FC, C_u, C_c, D_{50}\right) \quad (1)$$

where W, J/m^3, is measured strain energy density required for triggering liquefaction (capacity energy). W is the accumulative area of stress–strain loops up to the liquefaction triggering (see Figure 1). The input variables used to develop the capacity energy predictive correlations are as listed below:

$s\,?_{mean}$ (kPa): Soil initial effective mean confining pressure

D_r (%): Initial relative density after consolidation

FC (%): Percentage of fines content

C_u: Coefficient of uniformity

C_c: Coefficient of curvature

D_{50} (mm): Mean grain size

$s\,?_{mean}$ and D_r represent the soil initial density, and FC, C_u, C_c and D_{50} are grain size characteristics of soils. The significant influence of $s\,?_{mean}$ and D_r in determining W is well understood [23, 24]. The grain size distribution has traditionally been identified as one of the most important factors affecting the liquefaction characteristics of sands [3, 25]. The strong effect of FC, C_u, C_c and D_{50} to determine W was previously demonstrated by a few researchers [25]. As expected, coarser soils require higher unit energy for liquefaction than finer soils. C_u and C_c consider the influence of particle size range as well as symmetry and shape of the gradation curve on the unit energy required for liquefaction. Figueroa et al. [25] claimed that both C_u and C_c have significant influence in the energy per unit volume for liquefaction. After developing and controlling several correlations with different combinations of the input parameters, six GP/SA correlations were selected and presented as the optimal models. Variety of the input combinations in the presented correlations provides the possibility of using a specific correlation considering the available field data in future studies. In order to conduct a comparison between this study and the models found in the literature, the number of inputs to build the GP/SA-based correlations was reduced to two parameters ($s\,?_{mean}$, D_r). These parameters are the most widely used parameters in the available energy-based pore pressure build-up models for the liquefaction assessment. Table 2 summarizes the considered (optimal) combinations of the input parameters for the development of the correlations. Correlation coefficient (R), mean squared error (MSE) and mean absolute error (MAE) were used to evaluate the capabilities of the proposed correlations.

Table 1. Combinations of the input parameters used for the development of the correlations

Combination #	Parameters
1	s'_{mean}, D_r , FC, C_u, D_{50}, C_c
2	s'_{mean}, D_r , FC, C_u, D_{50}
3	s'_{mean}, D_r , FC, C_u
4	s'_{mean}, D_r , FC, D_{50}
5	s'_{mean}, D_r , FC
6	s'_{mean}, D_r

3.1. Experimental database

A comprehensive database of previously published cyclic tests was collected from the literature in order to develop general and well-built models for various types of soils with various initial states. The database consists of 224 cyclic triaxial [2, 26], 61 cyclic torsional shear [23, 27], 6 cyclic simple shear (VELACS project) [28], and 18 liquefaction triggering centrifuge [29] tests data. These are a total of 309 cyclic triaxial, torsional, simple shear and centrifuge ground level liquefaction element tests on Monterey, Northridge, Yatesville, Reid Bedford, LSFD, LSI-30, Toyoura and Nevada 40% clean and silty sands. Parts of these data have previously been presented by Baziar and Jafarian [9]. The database includes the measurements of several variables such as $s\,?_{mean}$ (kPa), D_r (%), FC (%), C_u, C_c, D_{50} (mm) and W (J/m^3). Furthermore, the database contains data of some element tests under random loading. The criteria for liquefaction triggering (failure) is initial liquefaction (r_u = 1) or double amplitude of strain of 5% (ε_{DA} = 5%), whichever occurs first. For the GP/SA and tree-based GP analyses, the data sets were randomly divided into training and testing subsets. In order to obtain a consistent data division, several combinations of the training and testing sets were considered. The selection was such that the maximum, minimum, mean and standard deviation of parameters were consistent in training and testing data sets. Out of the 309 data, 249 data were used as the training data and 60 data were taken for the testing of the generalization capability of the models. Although normalization is not strictly necessary in the GP-based analyses, better results are usually reached after normalizing the variables. This is mainly due to influence of unification of the variables, no matter their range of variation. Thus, both input and output variables were normalized between 0 and 1. Selection of the optimal method for normalizing the data was based on controlling several normalization methods [60]. The ranges, normalized forms, and statistics of different input and output parameters involved in the model development are given in Table 2.

Table 2. Descriptive statistics of the variables used in the development of the correlations

	Inputs						Output
Parameter	$s\,?_{mean}$ (kPa)	D_r (%)	FC (%)	C_u	C_c	D_{50} (mm)	$Log\ (W)$ (J/m^3)
Minimum	27.90	-37.15	0	1.57	0.74	0.03	2.48
Maximum	294.00	105.05	100	5.88	1.61	0.46	4.56
Range	266.10	142.20	100	4.31	0.87	0.43	2.08
Standard Deviation	31.163	32.660	25.631	1.079	0.233	0.125	0.454
Sample Variance	971.132	1066.696	656.926	1.164	0.054	0.016	0.206
Kurtosis	16.113	-0.012	2.875	3.847	1.294	-0.997	-0.105
Skewness	2.094	-0.822	1.831	2.061	1.396	0.443	0.679
Normalized Form	$s\,?_{mean}$ / 300	(D_r+40) / 150	(FC+40) / 150	C_u / 6	C_c / 2	D_{50} / 0.5	$Log\ (W)$ / 5

3.2. Correlation development using GP/SA

Six GP/SA-based correlations were selected and presented as the best models considering different combinations of $s ?_{mean}$, D_r, FC, C_u, C_c, and D_{50}. Various parameters involved in the GP/SA predictive algorithm are shown in Table 3. Several runs were conducted considering different values for the GP/SA parameters. The proper number of temperature levels depends on the number of possible solutions. It sets the number of temperature levels that the GP/SA algorithm uses until the run is terminated. Number of iterations per temperature level sets the number of times a new child program is created from the parent program at each temperature level. In order to find models with minimum error, each run was performed with large numbers of temperature levels and iterations. The program was run until no significant minimization of error was observed through the run. Three levels were set for the number of temperature levels parameter and two levels were used for the number of iterations. Temperature in the SA algorithm is just a number that controls the probability that a mutated child program will replace the parent program. Start and stop temperatures are respectively the values that a program uses for temperatures at the first and last temperature levels in a run. The initial and maximum program size parameters directly influence the size of the search space and the number of solutions explored within the search space. These parameters are measured in bytes. Two optimal values were set for the maximum program size as tradeoffs between the running time and the complexity of the evolved solutions.

Table 3. Parameter settings for the GP/SA algorithm

Parameter	Settings
Number of temperature levels	9000, 11000, 12000
Number of iterations per temperature level	1000, 1500
Start temperature	5
Stop temperature	0.01
Crossover rate (%)	50, 95
Homologous crossover (%)	95
Probability of randomly generated parent in crossover (%)	99
Offspring choice rate (%)	50
Replacement scaling factor	1
Maximum program size	128, 256
Initial program size	80
Function set	+, -, ×, /, v, sin, cos, tan

It is notable that the crossover rate parameter in the GP/SA algorithm sets the balance between the uses of the search operators (crossover and mutation). A value of 50% means that 50% of time the used search operator will be the crossover operator. The mutation operator will therefore be employed in the other 50% of time by the GP/SA algorithm [21]. Two levels were considered for the crossover rate. The squared error function was adopted as the fitness function. The values of the other involved parameters were selected based on some previously suggested values [e.g., 19] and also after performing many preliminary runs and checking the performance. There are 3×2×2×2= 24 different combinations of the parameters. All of these combinations were tested and 3 replications for each combination were performed which makes 72 runs for each input combination. Therefore, the total number of runs was equal to 72×6 (combination of the input variables) = 432. The GP/SA algorithm was implemented using the Discipulus software.

3.2.1. GP/SA-based correlations for the capacity energy prediction

The GP/SA-based prediction equations for the strain energy density required for triggering liquefaction, W (J/m^3), are as given below:

$$Log(W)=2(C_{u,n}-D_{r,n})^6+C_{c,n}\cos(C_{u,n}C_{c,n})s'^3_{mean,n}+\cos(\cos(D_{50,n}FC_n^{0.5}))+\sin((D_{r,n}-s'_{mean,n})\cos(C_{u,n}))\sin(s'^2_{mean,n}) \tag{2}$$

$$Log(W)=0.5+s'_{mean,n}D_{r,n}D_{50,n}+FC_n((D^6_{r,n})((s'_{mean,n}D_{r,n})\cos(C_{u,n}))) \tag{3}$$

$$Log(W)=0.05+8D^3_{r,n}/(s'_{mean,n}FC_n+7)^3+\cos(FC_n)^3(D_{r,n}-C_{u,n})^3+\cos(\cos(\cos(\cos(s'^{0.5}_{mean,n})^2)))^2 \tag{4}$$

$$Log(W)=\cos(\cos(FC_nD_{50,n}))+s'_{mean,n}D_{r,n}D_{50,n} \tag{5}$$

$$Log(W)=0.5+(s'_{mean,n}(FC_n-2D_{r,n}+2s'_{mean,n})-s'_{mean,n})^2 \tag{6}$$

$$Log(W)=0.5+0.47s'^{0.5}_{mean,n}D_{r,n} \tag{7}$$

and $\sigma?_{mean,n}$, $D_{r,n}$, FC_n, $C_{u,n}$, $C_{c,n}$ and $D_{50,n}$ are the input parameters in normalized forms shown in Table 2. Comparisons of the experimental versus predicted liquefaction capacity energy are shown in Figure 3.

3.3. Correlation development using traditional GP

A tree-based GP analysis was performed to compare the hybrid GP and SA technique, called GP/SA, with a classical GP approach. After developing and controlling several models with different combinations of the input parameters, the best tree-based GP correlation was selected and presented as the optimal model. Various parameters involved in the traditional GP predictive algorithm are shown in Table 4. The parameters were selected based on some previously suggested values and also after a trial and error approach. A tree-based GP software, GPLAB [30] in conjunction with subroutines coded in MATLAB was used in this study.

The formulation of W in terms of $s\,?_{mean}$, D_r, FC, C_u and $D_{50,}$ for the best result by the tree-based GP analysis, is as given below:

$$Log(W)=\frac{4}{(((7-(s'_{mean,n}+D_{r,n}+D^2_{50,n}))-(s'_{mean,n}(D_{r,n}-FC_n)))+C_{u,n})} \tag{8}$$

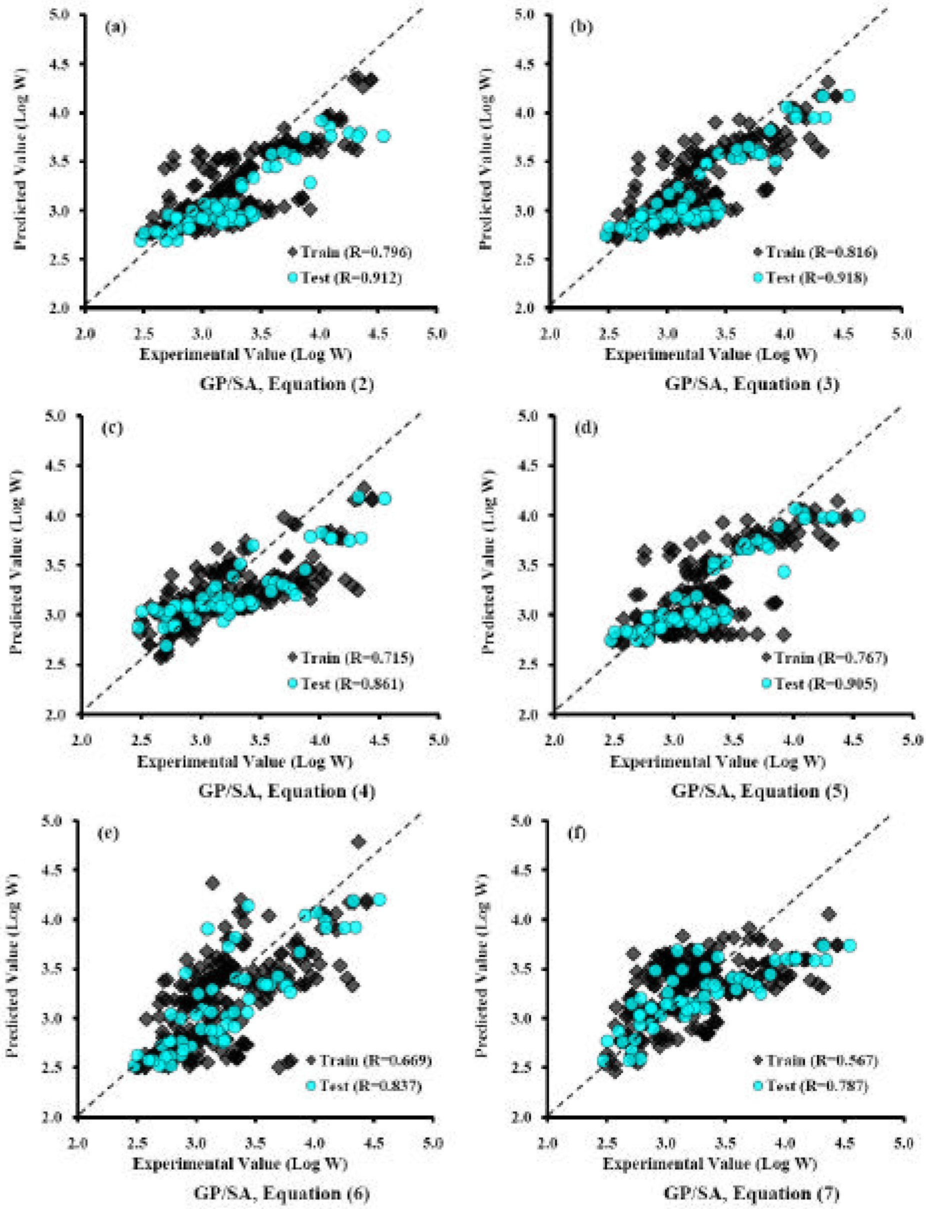

Figure 3. Experimental versus predicted capacity energy using the GP/SA correlations

Table 4. Parameter settings for the tree-based GP algorithm

Parameter	Settings
Function set	+, -, ×, /, sin, cos
Population size	200-1000
Maximum tree depth	10
Total generations	4000
Initial population	Ramped half-and-half
Sampling	Tournament
Expected no. of offspring method	Rank 89
Fitness function error type	linear error function
Termination	Generation 40
Minimum probability of crossover	0.1
Minimum probability of mutation	0.1
Real max level	30
Survival mechanism	Keep best

4. PERFORMANCE ANALYSIS OF THE ENERGY-BASED CAPACITY ENERGY CORRELATIONS

Different correlations were obtained for the liquefaction resistance assessment of sand–silt mixtures in terms of strain energy. Performance statistics of different correlations are summarized in Table 5. Since the other previously developed energy-based pore pressure build-up models need calibration parameters, it was not possible to evaluate their performance on the present database. Based on a logical hypothesis [31], if a model gives R > 0.8, and the error values (e.g., MSE and MAE) are at the minimum, there is a strong correlation between the predicted and measured values. It can be observed from Table 5 that the majority of the GP/SA models with high R and low MSE and MAE values are able to predict the target values with an acceptable degree of accuracy. The performance of the models on the training and testing data suggests that they have both good predictive abilities and generalization performance. However, in most cases, the results for the testing data are better than those for the training data sets. The results indicate that the GP/SA-based correlations with six, five, four, and three inputs significantly outperform those developed using two inputs. The above results imply the necessity of using more than two input variables (*s* ?$_{mean}$, D_r, *FC*, C_u, C_c, D_{50}) for the performed analyses. As it is seen, the GP/SA-based correlations provide considerably better results compared to the regression models [23, 24] found in the literature. Moreover, the proposed formulas using *s* ?$_{mean}$ and D_r as inputs perform superior than the available linear models using the same input variables. It can also be seen that the best GP/SA correlations (Equations (2) and (3)) have noticeably better performance than the optimal tree-based GP correlation. This indicates that applying the SA strategy to the GP process (GP/SA) has improved the efficiency of the traditional GP.

Table 5. Performance statistics of the energy-based correlations for the liquefaction assessment

Model	Train			Test			All Elements		
	R	MSE	MAE	R	MSE	MAE	R	MSE	MAE
Inputs: s ?$_{mean}$, D_r, FC, C_u, C_c, D_{50}									
GP/SA, Equation (2)	0.796	0.075	0.196	0.912	0.054	0.187	0.826	0.069	0.190
Inputs: s ?$_{mean}$, D_r, FC, C_u, D_{50}									
GP/SA, Equation (3)	0.816	0.066	0.199	0.918	0.045	0.174	0.838	0.062	0.188
Tree-Based GP, Equation (8)	0.773	0.083	0.228	0.896	0.066	0.210	0.800	0.080	0.224
Inputs: s ?$_{mean}$, D_r, FC, C_u									
GP/SA, Equation (4)	0.715	0.099	0.250	0.861	0.079	0.239	0.748	0.095	0.248
Inputs: s ?$_{mean}$, D_r, FC, D_{50}									
GP/SA, Equation (5)	0.767	0.085	0.217	0.905	0.049	0.177	0.797	0.078	0.209
Inputs: s ?$_{mean}$, D_r, FC									
GP/SA, Equation (6)	0.669	0.142	0.295	0.837	0.091	0.247	0.705	0.132	0.286
Inputs: s ?$_{mean}$, D_r									
GP/SA, Equation (7)	0.567	0.138	0.310	0.787	0.104	0.259	0.606	0.132	0.300
Liang [23]	–	–	– –	–	–	–	0.560	0.211	0.380
Liang [23]	–	–	– –	–	–	–	0.344	0.274	0.387
Dief and Figueroa [24]	–	–	– –	–	–	–	0.588	0.385	0.477
Dief and Figueroa [24]	–	–	– –	–	–	–	0.414	0.274	0.386

Furthermore, new criteria recommended by Golbraikh and Tropsha [32] were checked for the external verification of the GP/SA correlations on the testing data sets. It is suggested that at least one slope of regression lines (k or k') through the origin should be close to 1. Also, the performance indexes of m and n should be lower than 0.1. Recently, Roy and Roy [33] introduced a confirm indicator of the external predictability of models (R_m). For $R_m > 0.5$, the condition is satisfied. Either the squared correlation coefficient (through the origin) between predicted and experimental values (Ro^2), or the coefficient between experimental and predicted values (Ro'^2) should be close to 1. The considered validation criteria and the relevant results obtained by the models are presented in Table 6. As it is seen, the derived correlations satisfy most of the required conditions. With the exception of Equations (2) and (3), the other equations fail to satisfy the R_m criterion ($R_m >$ 0.5). However, the validation phase ensures the proposed correlations are valid, have the prediction power and are not chance correlations.

Table 6. Statistical parameters of the GP/SA correlations for the external validation

Formula	Condition	GP/SA Equation (2)	GP/SA Equation (3)	GP/SA Equation (4)	GP/SA Equation (5)	GP/SA Equation (6)	GP/SA Equation (7)
$k=\frac{\sum_{i=1}^{n}(h_i \times t_i)}{h_i^2}$	0.85< K<1.15	1.054	1.023	1.018	1.022	1.028	1.019
$k'=\frac{\sum_{i=1}^{n}(h_i \times t_i)}{t_i^2}$	0.85< K'<1.15	0.945	0.9743	0.975	0.975	0.965	0.972
$m=\frac{R^2-Ro^2}{R^2}$	m < 0.1	-0.01	-0.16	-0.33	-0.19	-0.38	-0.59
$n=\frac{R^2-Ro'^2}{R^2}$	n < 0.1	0.09	-0.14	-0.26	-0.17	-0.35	-0.46
$R_m = R^2 \times (1-\sqrt{\lvert R^2-Ro^2 \rvert})$	$R_m > 0.5$	0.78	0.53	0.38	0.49	0.34	0.25
$Ro^2 = 1-\frac{\sum_{i=1}^{n}(t_i-h_i^o)^2}{\sum_{i=1}^{n}(t_i-\overline{t_i})^2}$, $h_i^o = k \times t_i$		0.870	0.977	0.985	0.979	0.964	0.983
$Ro'^2 = 1-\frac{\sum_{i=1}^{n}(h_i-t_i^o)^2}{\sum_{i=1}^{n}(h_i-\overline{h_i})^2}$, $t_i^o = k' \times h_i$		0.764	0.959	0.934	0.961	0.949	0.906

5. VALIDITY VERIFICATION OF THE GP/SA-BASED CORRELATIONS

To verify the validity of the best GP/SA correlation, it was applied to some downhole array data of real earthquake case histories and some liquefaction centrifuge tests. It is well-known that a sand deposit would experience liquefaction if the strain energy imparted by earthquake to the sand layer (demand energy) is higher than the capacity energy of the liquefied soil [23]. This theory was used to examine the best GP/SA correlations (Equation (3)). Zeghal et al. [34] and Elgamal et al. [35] studied the downhole earthquake response at Port Island, Kobe site during 1995 Hyogoken-Nanbo earthquake and Lotung site, Taiwan 1986 LSST 16 earthquake. A simple identification technique was developed to estimate the shear stress-strain histories within the soil layers, directly from the free-field downhole acceleration records. This nonparametric identification procedure was firstly proposed for shake-table studies [36]. Zeghal et al. [37] used the above mentioned technique to propose average shear stress–strain loop of a fully liquefied portion of a centrifuge test (VELACS project, Davis Model 6). Average shear stress and strain loops at fully liquefied portions of the modeled dykes and slopes in C-Core project were developed using a simple identification technique proposed by Zeghal and Elgamal [38]. The areas of shear stress–strain loops for Kobe and Lotung downhole array sites, Davis model 6 centrifuge test and also C-Core project centrifuge data, were accumulatively calculated as real demand energy imparted into the soil.

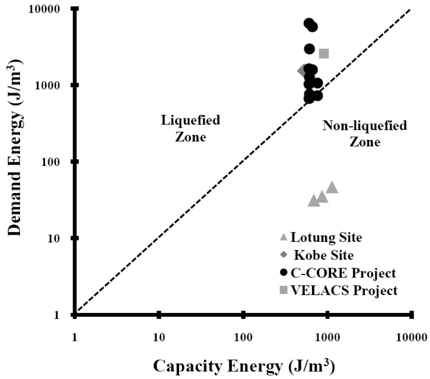

Figure 4. Comparison between the actual demand energy and predicted capacity energy by GP/SA

Figure 4 depicts the calculated demand energy versus the capacity energy predicted by the best GP/SA-based formulas. These figures demonstrate that the capacity energy values predicted by the proposed formulas are lower than their corresponding demand energy values for liquefied case histories of Kobe site, Davis model 6 and C-Core project centrifuge tests. The Lotung downhole array site did not experience liquefaction in the 1986 LSST 16 earthquake. As can be seen in Figure 4, the capacity energy values predicted by the GP/SA-based formula for various depths of Lotung site are higher than their corresponding demand energy values. It is evident that the proposed formula yields reasonable results for all the liquefied and non-liquefied cases histories.

6. SENSITIVITY ANALYSIS

Sensitivity analysis is of utmost concern for selecting the important input variables. The contribution of each input parameter in the GP/SA correlations was evaluated through a sensitivity analysis. In order to evaluate the importance of the input parameters, their frequency values were obtained. A frequency value equal to 1.00 for an input indicates that this input variable has been appeared in 100% of the best thirty programs evolved by GP/SA. $\sigma?_{mean}$ and D_r represent initial density of soils and, therefore, they were categorized into one group referred as Intergranular

Contact Density. FC was considered as a single category which controls potential of pore pressure build-up. C_u, C_c and D_{50} are the grain size distribution parameters and were categorized into a separate group as Textural Properties.

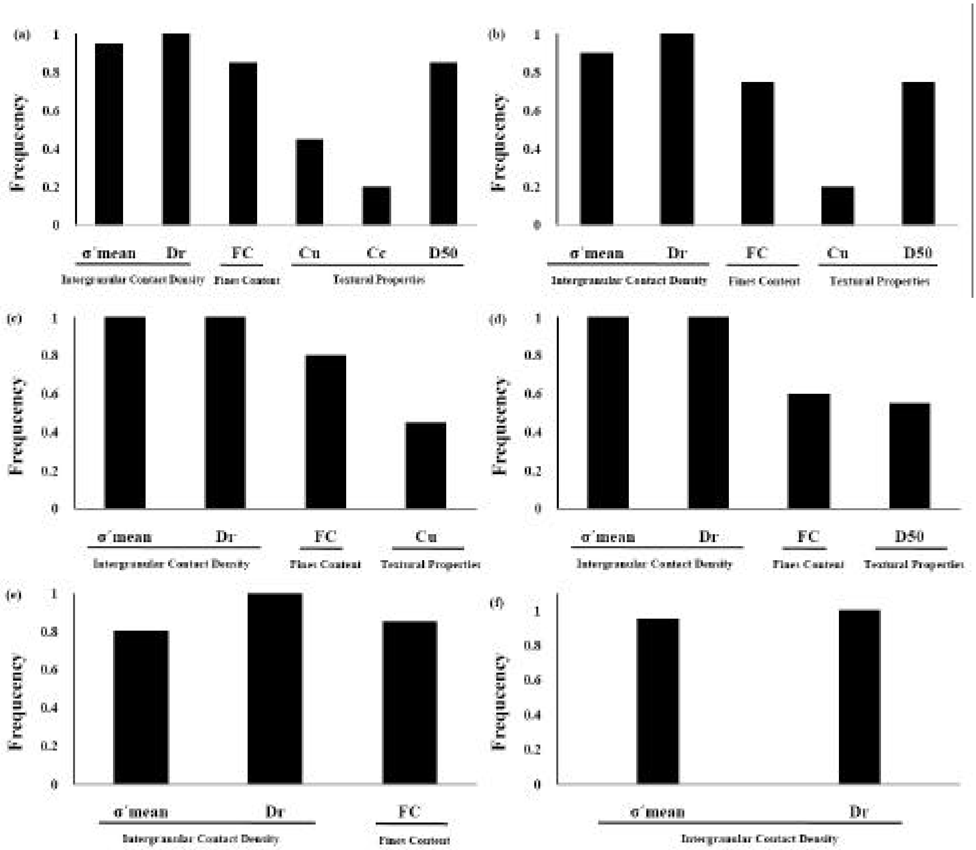

Figure 5. Contributions of the predictor variables in the GP/SA-based correlations

The frequency values of the input parameters of the models are presented in Figure 5. According to this figure, the capacity energy in the GP/SA models is more sensitive to D_r and $\sigma?_{mean}$ compared with the other parameters. As mentioned before, $\sigma?_{mean}$ and D_r are the most widely used parameters incorporated directly in majority of the previous conventional models. In the models using D_r and $\sigma?_{mean}$ as the input parameters (Figure 7(f)), the capacity energy is more sensitive to D_r than $\sigma?_{mean}$. As it is seen, the results obtained by GP/SA are in agreement with each other. The essential observation from the results of the sensitivity analysis is that the capacity energy is more sensitive to FC in comparison with C_u, C_c and D_{50}. While FC exerts dominant influence on the capacity energy variations, it has not been directly incorporated in majority of the previous models. In this context, there are earlier findings that are in agreement with this observation [9].

7. CONCLUSIONS

New empirical correlations were derived to estimate the amount of strain energy required up to liquefaction triggering using the GP/SA paradigm. Different combinations of the influencing parameters ($s?_{mean}$, D_r, FC, C_u, C_c, D_{50}) were considered for the development of the proposed correlations. It was observed that the proposed GP/SA correlations give reliable estimates of the capacity energy values of a soil system to experience liquefaction triggering. The validity of the correlations was tested for a part of test results beyond the training data domain. Furthermore, the correlations efficiently satisfy the conditions of different criteria considered for their external validation. Due to high nonlinearity in liquefaction development, the proposed nonlinear models produce considerably better outcomes over the existing linear energy-based pore pressure build-up models. Also, the best GP/SA correlations are found to be more accurate in comparison with the best correlation evolved by the traditional GP method. Unlike majority of the previous studies on the constitutive modeling of liquefaction, the proposed correlations simultaneously take into account the role of several important factors. The accuracy of the results strongly confirms that $s?_{mean}$, D_r, FC, C_u, C_c, and D_{50} can be regarded as efficient representatives of the liquefaction behavior. The correlations developed using different combinations of $s?_{mean}$, D_r, FC, C_u, C_c, and D_{50} considerably outperform those using $s?_{mean}$ and D_r. This implies the necessity of incorporating more soil initial parameters into the analysis rather than using only the most widely used parameters of $s?_{mean}$ and D_r. The data recorded during some real earthquakes at Port Island Kobe and Lotung sites plus some available centrifuge tests data were used for further validation the best GP/SA-based correlations. The verification phase confirms the efficiency of the models for their general application to the assessment of the strain energy at the onset of liquefaction.As the other researchers have mentioned, the sensitivity analysis results indicate that the capacity energy is more sensitive to D_r and $s?_{mean}$. Another finding from the results of the sensitivity study is that FC is also more effective to explain the variations of the capacity energy compared with the other soil parameters. Despite the important role of FC in the liquefaction potential assessment, most of the previous models have not directly taken into account the effect of this parameter. Some researchers claimed that it is more realistic to draw a relationship between the liquefaction resistance of sands and any of the size (i.e. D_{10}, D_{30} or D_{60}) (e.g., [80, 86]). Accordingly, further research can focus on developing correlations between the capacity energy in liquefiable soils and D_{10}, D_{30} or D_{60}, instead of the grading characteristics (i.e. C_u and C_c).

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Dynamic modeling of the viaduct subjected on traffic actions

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Summary

The article proposes an approach of six degrees dynamic model of a rigid-solid with some types of symmetries. These symmetries lead to simplified mathematical models, which are more easily to solve. If the rigid-solid is symmetrical beared by triorthogonal elastic links, the mathematical model becomes still simple and the vibrations are decoupled into four subsystems of movements: side slipping and rolling, forward motion and pitching, lifting motion, gyration. There are two case study of modal analysis: for a viaduct with five arches made from reinforced concrete „U" beam and for an arch (between two piers of the viaduct) made from four reinforced concrete „U" beam.

KEYWORDS: decoupled vibration, eigenvalues, modal calculus, structural symmetry, reinforced concrete bridges vibrations.

1. INTRODUCTION

The mathematical modeling uses the physical model of the rigid solid with six degrees of freedom (6DOF) with a finite number of viscous-elastic bearings. Dimensional and inertial characteristics of the rigid solid and rheological characteristics of the bearings (stiffness and damping) can be experimentally determined by direct measurements and by static and/or dynamic testing. According to (7), the differential equations of the movements of the rigid solid with viscous-elastic bearings are coupled by stiffness and damping coefficients. The system of the equations can be write as follows:

$$\underline{A}\,\underline{\underline{\ddot{q}}} + \underline{B}\,\underline{\underline{\dot{q}}} + \underline{C}\,\underline{\underline{q}} = \underline{\underline{f}} \ , \tag{1}$$

where $\underline{A}$ is the inertia matrix; $\underline{B}$ is the viscous damping matrix (damping coefficients); $\underline{C}$ is the elasticity matrix (stiffness coefficients); $\underline{\underline{q}} / \underline{\underline{\dot{q}}} / \underline{\underline{\ddot{q}}}$ are generalized displacements / velocities / accelerations vector and $\underline{\underline{f}}$ is the generalized forces vector.

If the damping coefficients are small, the differential equations system becomes:

$$\underline{\underline{A}}\ddot{\underline{\underline{q}}} + \underline{\underline{C}}\underline{\underline{q}} = \underline{\underline{f}} \tag{2}$$

Considering the rigid solid no perturbated, the system of differential equations becomes:

$$\underline{\underline{A}}\ddot{\underline{\underline{q}}} + \underline{\underline{C}}\underline{\underline{q}} = \underline{\underline{0}} \ , \tag{3}$$

where $\underline{\underline{0}}$ is the null vector (where all coefficients are zero).

If the Cartesian coordinates axis system is central and principal, the quadratic 6×6 inertia matrix becomes diagonal

$$\underline{A} = DIAG\lfloor m, m, m, J_x, J_y, J_z \rfloor \ , \tag{4}$$

where m is the rigid solid mass and J_x, J_y, J_z are the principal inertia moments.

2. THE RIGID SOLID WITH STRUCTURAL SYMMETRIES. MODAL ANALYSIS

Considering that the rigid solid has an vertical axis of symmetria (mass distribution, geometrical configuration, bearings disposal) and the coordinate system is central and principal, the inertia matrix is diagonal. If the elastic bearing system of the rigid solid is composed from n supports with triorthogonal stiffness (k_{ix}, k_{iy}, k_{iz}) like in figure 1, with the position done by the coordinates $M_i(x_i, y_i, z_i)$ $i = \overline{1,n}$, the elasticity matrix becomes:

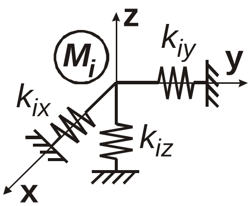

Figure. 1 Elastic triorthogonal bearing

$$\underline{C}=\begin{bmatrix} \sum k_{ix} & 0 & 0 & 0 & \sum k_{ix}z_i & 0 \\ 0 & \sum k_{iy} & 0 & -\sum k_{iy}z_i & 0 & 0 \\ 0 & 0 & \sum k_{iz} & 0 & 0 & 0 \\ 0 & -\sum k_{iy}z_i & 0 & \sum\left(k_{iy}z_i^2+k_{iz}y_i^2\right) & 0 & 0 \\ \sum k_{ix}z_i & 0 & 0 & 0 & \sum\left(k_{iz}x_i^2+k_{ix}z_i^2\right) & 0 \\ 0 & 0 & 0 & 0 & 0 & \sum\left(k_{ix}y_i^2+k_{iy}x_i^2\right) \end{bmatrix} \quad (5)$$

As the inertia matrix is diagonal, the coefficients outside the main diagonal of the elasticity matrix $\underline{C}$ are the coupling terms of the equations of the system (3). Because there are only four non-zero stiffness coefficients ($c_{15}\equiv c_{51}$ and $c_{24}\equiv c_{42}$), the free movements of the rigid solid are decoupled into four subsystems with coupled vibrations. The subsystems with coupled motion equations are as follows:

a) subsystem $\left(X,\varphi_y\right)$ - side slip movement coupled with rolling movement

$$\begin{cases} m\ddot{X}+X\sum k_{ix}+\varphi_y\sum k_{ix}z_i=0 \\ J_y\ddot{\varphi}_y+X\sum k_{ix}z_i+\varphi_y\sum\left(k_{iz}x_i^2+k_{ix}z_i^2\right)=0 \end{cases} \quad (6)$$

b) subsystem $\left(Y,\varphi_x\right)$ - forward-back movement coupled with pitch movement

$$\begin{cases} m\ddot{Y}+Y\sum k_{iy}-\varphi_x\sum k_{iy}z_i=0 \\ J_x\ddot{\varphi}_x-Y\sum k_{iy}z_i+\varphi_x\sum\left(k_{iy}z_i^2+k_{iz}y_i^2\right)=0 \end{cases} \quad (7)$$

c) subsystem (Z) - up-down movement

$$m\ddot{Z}+Z\sum k_{iz}=0 \quad (8)$$

d) subsystem $\left(\varphi_z\right)$ - turning movement (gyration)

$$J_z\ddot{\varphi}_z+\varphi_z\sum\left(k_{ix}y_i^2+k_{iy}x_i^2\right)=0 \quad (9)$$

In order to determinate the natural frequencies and the eigenvalues we use the next notations:

- for the pulsations of the no coupled movements of translation

$$p_X=\sqrt{\frac{\sum k_{ix}}{m}} \quad p_Y=\sqrt{\frac{\sum k_{iy}}{m}} \quad p_Z=\sqrt{\frac{\sum k_{iz}}{m}} \quad (10a)$$

- for the pulsations of the no coupled movements of rotation

$$p_{\varphi_x} = \sqrt{\frac{\sum\left(k_{iy} z_i^2 + k_{iz} y_i^2\right)}{J_x}}$$

$$p_{\varphi_y} = \sqrt{\frac{\sum\left(k_{iz} x_i^2 + k_{ix} z_i^2\right)}{J_y}} \tag{10b}$$

$$p_{\varphi_z} = \sqrt{\frac{\sum\left(k_{ix} y_i^2 + k_{iy} x_i^2\right)}{J_z}}$$

- the dynamic coupling terms for the $\left(X, \varphi_y\right)$ and $\left(Y, \varphi_x\right)$ subsystems

$$\begin{cases} \alpha_1 = \dfrac{1}{m} \sum k_{ix} z_i \\ \alpha_2 = \dfrac{1}{J_y} \sum k_{ix} z_i \end{cases} \qquad \begin{cases} \beta_1 = -\dfrac{1}{m} \sum k_{iy} z_i \\ \beta_2 = -\dfrac{1}{J_x} \sum k_{iy} z_i \end{cases} \tag{11}$$

Considering the relations (10) and (11), the natural pulsations and the eigenvalues of the decoupled subsystems can be determinate with the next calculus formula:

a) for the subsystem $\left(X, \varphi_y\right)$

$$p_{1,2} = \sqrt{\frac{1}{2}\left[p_X^2 + p_{\varphi_y}^2 \mp \sqrt{\left(p_X^2 - p_{\varphi_y}^2\right)^2 + 4\alpha_1\alpha_2}\right]} \tag{12}$$

$$\mu_{1,2} = -\frac{1}{2\alpha_1}\left[p_X^2 + p_{\varphi_y}^2 \pm \sqrt{\left(p_X^2 - p_{\varphi_y}^2\right)^2 + 4\alpha_1\alpha_2}\right] \tag{13}$$

b) for the subsystem $\left(Y, \varphi_x\right)$

$$p_{3,4} = \sqrt{\frac{1}{2}\left[p_Y^2 + p_{\varphi_x}^2 \mp \sqrt{\left(p_Y^2 - p_{\varphi_x}^2\right)^2 + 4\beta_1\beta_2}\right]} \tag{14}$$

$$\mu_{3,4} = -\frac{1}{2\beta_1}\left[p_Y^2 + p_{\varphi_x}^2 \pm \sqrt{\left(p_Y^2 - p_{\varphi_x}^2\right)^2 + 4\beta_1\beta_2}\right] \tag{15}$$

3. MODAL ANALYSIS OF A BRIDGE MADE FROM REINFORCED CONCRETE

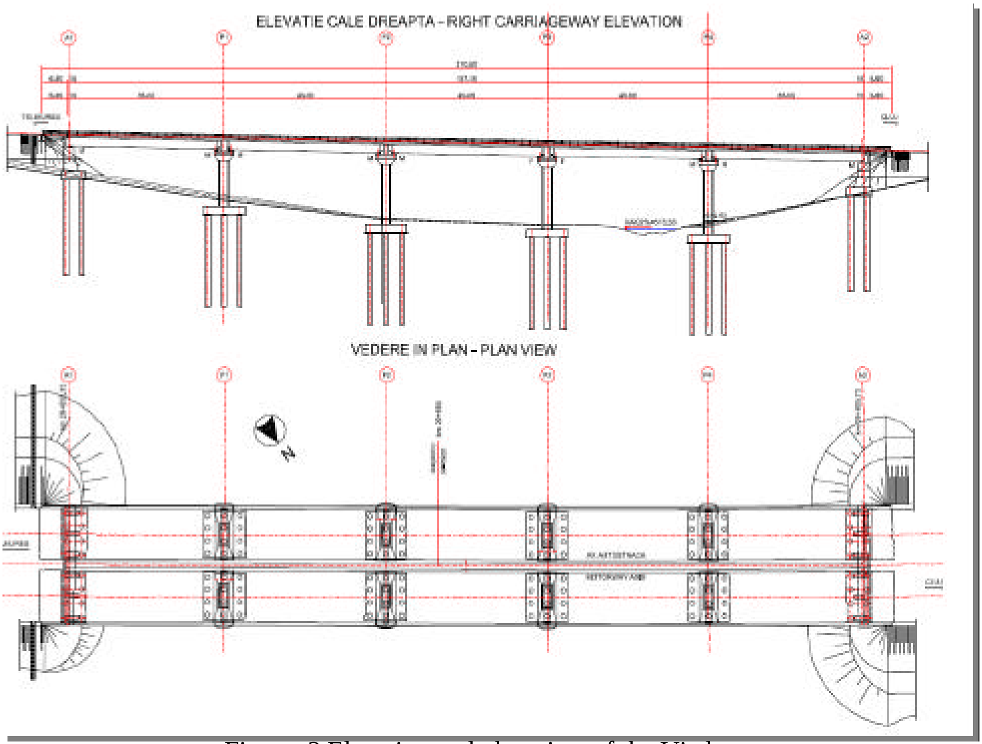

Figure. 2 Elevation and plan view of the Viaduct;
Romanian highway A3 - KM 29+602,75? KM 29+801,25

Figure 2 shows elevation and the plan view for a bridge made from twenty reinforced concrete beams jointed through a 300 mm thickness reinforced concrete plate. Each beam is beared on the piers and on the abutments of the bridge through four identically viscous-elastic supports made from neoprene; there a total numer of eighty neoprene bearings for the entire bridge. The simplified model of the bridge is shown in the figure 3.

In order to calculate the natural pulsations and frequencies and the eigenvalues of the bridge modeled as in the figure 2, the main characteristics are the next:

¦ Dimensions (as in detailed engineering drawings and/or measured):

?for “U” beams: $37100 \times 1700 / 3280 \times 2200$ lenght×width×height [mm]

?for the bridge: $200000 \times 13300 \times 2500$ lenght×width×height [mm]

- Stiffness of the neoprene bearings (experimental measurements):

$$k_{ix} \equiv k_x = 3{,}15 \times 10^6 \ N/m \quad i = \overline{1{,}80}$$

$$k_{iy} \equiv k_y = 3{,}15 \times 10^6 \ N/m \quad i = \overline{1{,}80}$$

$$k_{iz} \equiv k_z = 650 \times 10^6 \ N/m \quad i = \overline{1{,}80}$$

- Masses and inertia according to table 1 (calculated):

Table 1. Inertial characteristics (central and principal axis system)

Denomination		**Unit**	**Arch of the viaduct (4 beams)**	**Viaduct (20 beams)**
Mass m		kg	992,000	4,960,000
Products of inertia		Kg·m^2	$J_{xy} = J_{yz} = J_{zx} = 0$	
Moments of inertia	J_x	Kg·m^2	120.533×10^6	16.025×10^9
	J_y	Kg·m^2	15.133×10^6	73.270×10^6
	J_z	Kg·m^2	134.091×10^6	16.092×10^9

Table 2. Positions of the neoprene bearings

Bearing and coordinates [m]															
i	**x_i**	**y_i**	**z_i**	**i**	**x_i**	**y_i**	**z_i**	**i**	**x_i**	**y_i**	**z_i**	**i**	**x_i**	**y_i**	**z_i**
1	-5,5	-98,05	-1,45	**21**	1,1	-58,05	-1,45	**41**	-5,5	18,05	-1,45	**61**	1,1	58,05	-1,45
2	-4,4	-98,05	-1,45	**22**	2,2	-58,05	-1,45	**42**	-4,4	18,05	-1,45	**62**	2,2	58,05	-1,45
3	-2,2	-98,05	-1,45	**23**	4,4	-58,05	-1,45	**43**	-2,2	18,05	-1,45	**63**	4,4	58,05	-1,45
4	-1,1	-98,05	-1,45	**24**	5,5	-58,05	-1,45	**44**	-1,1	18,05	-1,45	**64**	5,5	58,05	-1,45
5	1,1	-98,05	-1,45	**25**	-5,5	-21,95	-1,45	**45**	1,1	18,05	-1,45	**65**	-5,5	61,95	-1,45
6	2,2	-98,05	-1,45	**26**	-4,4	-21,95	-1,45	**46**	2,2	18,05	-1,45	**66**	-4,4	61,95	-1,45
7	4,4	-98,05	-1,45	**27**	-2,2	-21,95	-1,45	**47**	4,4	18,05	-1,45	**67**	-2,2	61,95	-1,45
8	5,5	-98,05	-1,45	**28**	-1,1	-21,95	-1,45	**48**	5,5	18,05	-1,45	**68**	-1,1	61,95	-1,45
9	-5,5	-61,95	-1,45	**29**	1,1	-21,95	-1,45	**49**	-5,5	21,95	-1,45	**69**	1,1	61,95	-1,45
10	-4,4	-61,95	-1,45	**30**	2,2	-21,95	-1,45	**50**	-4,4	21,95	-1,45	**70**	2,2	61,95	-1,45
11	-2,2	-61,95	-1,45	**31**	4,4	-21,95	-1,45	**51**	-2,2	21,95	-1,45	**71**	4,4	61,95	-1,45
12	-1,1	-61,95	-1,45	**32**	5,5	-21,95	-1,45	**52**	-1,1	21,95	-1,45	**72**	5,5	61,95	-1,45
13	1,1	-61,95	-1,45	**33**	-5,5	-18,05	-1,45	**53**	1,1	21,95	-1,45	**73**	-5,5	98,05	-1,45
14	2,2	-61,95	-1,45	**34**	-4,4	-18,05	-1,45	**54**	2,2	21,95	-1,45	**74**	-4,4	98,05	-1,45
15	4,4	-61,95	-1,45	**35**	-2,2	-18,05	-1,45	**55**	4,4	21,95	-1,45	**75**	-2,2	98,05	-1,45
16	5,5	-61,95	-1,45	**36**	-1,1	-18,05	-1,45	**56**	5,5	21,95	-1,45	**76**	-1,1	98,05	-1,45
17	-5,5	-58,05	-1,45	**37**	1,1	-18,05	-1,45	**57**	-5,5	58,05	-1,45	**77**	1,1	98,05	-1,45
18	-4,4	-58,05	-1,45	**38**	2,2	-18,05	-1,45	**58**	-4,4	58,05	-1,45	**78**	2,2	98,05	-1,45
19	-2,2	-58,05	-1,45	**39**	4,4	-18,05	-1,45	**59**	-2,2	58,05	-1,45	**79**	4,4	98,05	-1,45
20	-1,1	-58,05	-1,45	**40**	5,5	18,05	-1,45	**60**	-1,1	58,05	-1,45	**80**	5,5	98,05	-1,45

¦ Position of the mass center **C** against the neoprene bearings (calculated): $h = 1454{,}4mm$

¦ Positions of the neoprene bearingson the viaduct (related to the centered coordinate system **Cxyz**) as in detailed engineering drawings – see table 2.

Using the relations (10), the natural pulsations p and the natural frequencies f of the uncoupled vibrations for the six degrees of dynamic freedom are shown in the table 3.

Table 3. Natural pulsations and frequencies (on the six degrees of dynamic freedom)

System	**Direction**	X	Y	Z	φ_x	φ_y	φ_z
Arch of the viaduct (4 beams)	p [rad/s]	7.13	7.13	102.39	167.67	97.83	11.30
	f [Hz]	1.13	1.13	16.30	26.69	15.60	1.80
Viaduct (20 beams)	p [rad/s]	7.13	7.13	102.39	105.49	97.83	7.34
	f [Hz]	1.13	1.13	16.30	16.79	15.60	1.17

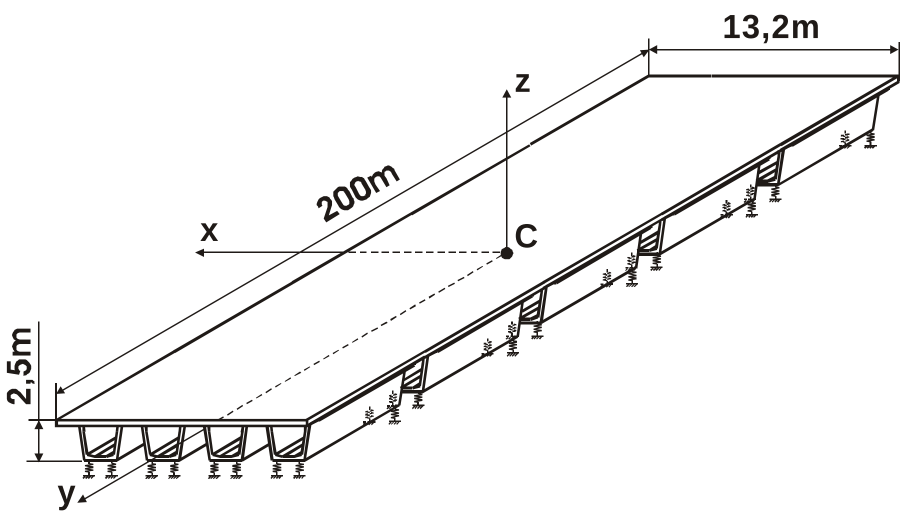

Figure. 3 The model of the bridge beared on eighty neoprene supports

The figures from table 4 show the values of the natural pulsations and frequencies and of the eigenvalues for the decoupled subsystems (with coupled movements) for a bridge section (arche) composed from four „U" beams as in figure 4 and figure 5. As it can see, there are the same values for pulsations and frequencies like in table 3. That means, the movements inside the subsystems (X, j_y) and (Y, j_x) are very weak coupled, almost uncoupled.

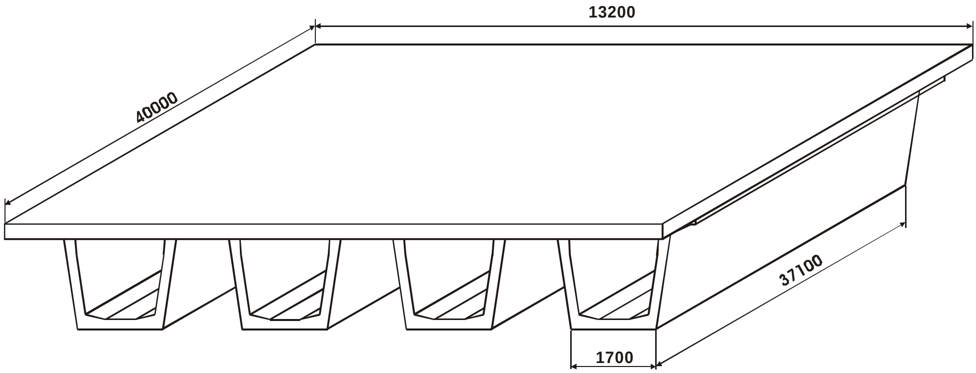

Figure. 4 The model of an arch of the viaduct

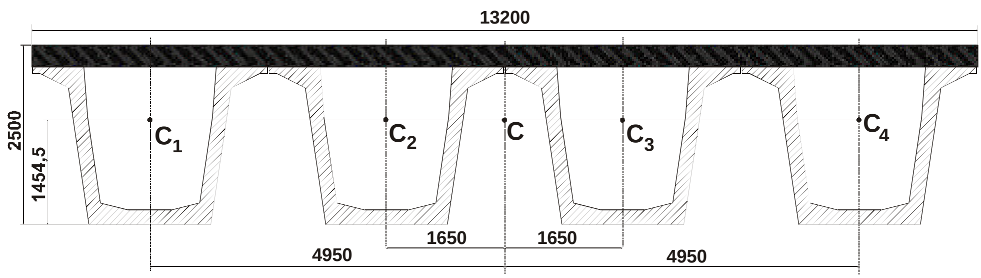

Figure. 5 The model of an arch of the viaduct (transversal section)

Table 4. Modal analyze for an arch (section) of the viaduct (decoupled subsystems)

Subsystem	Pulsations	Frequencies	Eigenvalues
(X, φ_y)	$p_1 = 7.13 rad/s$	$f_1 = 1.13 Hz$	$\mu_1 = 0.000509 rad/m$
	$p_2 = 97.83 rad/s$	$f_2 = 15.60 Hz$	$\mu_2 = -128.824 rad/m$
(Y, φ_x)	$p_3 = 7.13 rad/s$	$f_3 = 1.13 Hz$	$\mu_3 = -0.000002 rad/m$
	$p_4 = 167.67 rad/s$	$f_4 = 26.69 Hz$	$\mu_4 = 379.750 rad/m$
(Z)	$p_5 = p_Z = 102.39 rad/s$	$f_5 = f_Z = 16.30 Hz$	-
(φ_z)	$p_6 = p_{\varphi_z} = 11.30 rad/s$	$f_6 = f_{\varphi_z} = 1.80 Hz$	-

The figures from table 5 show the values of the natural pulsations and frequencies and of the eigenvalues for the decoupled subsystems (with coupled movements) for the entire bridge composed from five sections (arches) considered being identical as in figure 3. As for the arches, the movements inside the subsystems with coupled movements $\left(X, j_y\right)$ and $\left(Y, j_x\right)$ of the viaduct are very weak coupled, almost uncoupled.

Table 5. Modal analyze for the viaduct (decoupled subsystems)

Subsystem	Pulsations	Frequencies	Eigenvalues
$\left(X, \varphi_y\right)$	$p_1 = 7.13 rad/s$	$f_1 = 1.13 Hz$	$\mu_1 = 0.000509 rad/m$
	$p_2 = 97.83 rad/s$	$f_2 = 15.57 Hz$	$\mu_2 = -128.824 rad/m$
$\left(Y, \varphi_x\right)$	$p_3 = 7.13 rad/s$	$f_3 = 1.13 Hz$	$\mu_3 = -0.000002 rad/m$
	$p_4 = 105.49 rad/s$	$f_4 = 16.79 Hz$	$\mu_4 = 149.916 rad/m$
(Z)	$p_5 = p_Z = 102.39 rad/s$	$f_5 = f_Z = 16.30 Hz$	-
$\left(\varphi_z\right)$	$p_6 = p_{\varphi_z} = 7.34 rad/s$	$f_6 = f_{\varphi_z} = 1.17 Hz$	-

3. CONCLUSIONS

a) modeling a rigid solid with elastic or viscous-elastic bearings and symmetries (structural, inertial, bearings) lead to linear mathematical models more simple, with differential equations decoupled into subsystems easier to solve; in this case, we can highlight the influences of different kinds of characteristics (dimensions, masses, inertia, stiffness) on the dynamic parameters of the rigid solid (natural pulsations/frequencies, eigenvalues);

b) if the physical model of the rigid solid permits to chose a Cartesian coordinate system which is central and principal, then the differential equations of motion are coupled only by the coefficients outside of principal diagonal of elasticity matrix (elastic coupling of movements), eventually by the dissipation coefficients from the viscous damping matrix if they are significant;

c) comparing the values of the pulsations/frequencies from the tables 3, 4 and 5, we can say that the movements inside the subsystems are almost uncoupled on the "directions" ($X, Y, Z, \varphi_x, \varphi_y, \varphi_z$); also the values very small or very big of the eigenvalues can explain the quasidecoupling of the movements inside of the subsystems;

d) analyzing the values from table 4 (for the arches), we can find a group of three natural frequencies in the domain 1.1÷1.2 Hz, another one in the domain 15.6÷16.3 Hz and the 6-th frequency being much more bigger (26.69 Hz); this grouping of frequencies and the big differences between the values of domains' limits can be explained by the significant differences between the bearings stiffness on vertical axis **Cz** (compression effort) and on horizontal plane **xCy** (shear efforts);

e) analyzing the values from table 5 (for the entire bridge), we can find a group of three natural frequencies in the domain 1.1÷1.2 Hz and another three in the domain 15.6÷16.8 Hz; in this case of simulation, the pitch movement (φ_x) of the viaduct, which is almost decoupled from the forward-back movement (Y), has a natural frequency more smaller than the pitch movement of a single arch because of a bigger value of the moment of inertia J_x mainly.

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Use of nondestructive methods in the feasibility study stage to identify the utilities within the path of future road infrastructure works

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Summary

Buried utilities and cables represent a major risk for any construction work on road infrastructure as well as for factors involved in its design, implementation, operation and post-use.

Getting the most accurate information about the location of various buried utilities, cables, pipes, etc. is an absolute necessity before starting any excavation in order to achieve the construction of road communication routes.

Although in the authorization of construction works process is the sole responsibility of the beneficiary to obtain the planning certificate , which involves obtaining approvals and agreements from various authorities including the owners of utilities, however, currently there are cases where a part of them are not reported, and not included in the plans. This can result in most cases to the damage of the utilities with financial repercussions, but also inconveniences to users.

For this reason the use of nondestructive methods for detecting, locating and reproducing the spatial position of the utilities on detailed maps are desirable and may lead to substantial cost reductions.

Minimizing costs and reducing of risks is the goal of any beneficiary who wishes to obtain profitable investments.

Detection and tracking of utilities before starting the execution of communication route must be included as a mandatory report on the feasibility study, which is recommended by the analysis that was the basis of this paper.

KEYWORDS: utility networks, non-destructive methods, magnetometry, georadar method (GPR), ground penetrating radar

1. INTRODUCTION

1.1. General issues

After analyzing the situation regarding the location of utility networks on the path of future road infrastructure and the consultation of approvals / agreements of principle obtained from the owners of utilities is noted that in some areas, it is required the relocation / protection works of these networks. This stems from the fact that the aerial or underground networks are located either in the immediate vicinity of the road or in the roadway.

In Government Program for 2009-2012 at Chapter 12 - Transport infrastructure one of the main lines of action related to road transport is to ammend the legislation regarding relocation of utilities for execution of motorways and ring roads, by involving the owners of these utilities, pursuing compliance with deadlines and minimal relocation costs.

Changes of technical solutions due to problems encountered with the relocation of utilities lead to delay of implementation and increase of costs.

In the composition of costs and execution period of the construction should be included also issues linked to underground utilities relocation arising additional costs. Those costs are even higher as the road is closer to towns.

Achieving constructive technical, geometric and functional characteristics of road may lead to relocating works and, although their identification is performed together with the owners of networks on topographical plans (situation plans, longitudinal and transverse sections) equipped with utility networks, however, on field, when starting the excavations for executing the earthworks additional networks are detected .

1.2. Structure and size of utility networks

Technical and urban infrastructure represent all the ansamble of constructions, technological equipment, functional equipment and specific facilities including various networks of utilities (water, sewer, gas, termic and electric energy, telecommunications, data communications and Internet, etc..).

The components of a drinkable water distribution system are:

- tubes and pipes for transporting drinkable water from the inlet pipes or pumping stations to the points of connection to consumers (including service systems, main and secondary arteries of distribution)
- catchings, pipelines and inlets (pipe which connects a main and a secondary duct)

- treatment plants of raw water, potable water storage tanks.

A public sewer system consists of:

- tubes for collection and disposal of domestic and industrial waste water and pluvial, from connection locations to the point of discharge of wastewater into a natural emissary (including service systems, main and secondary collector sewers).

Natural gas distribution infrastructure consists of all current transport pipelines (excluding main pipelines) from pressure regulating stations and gas delivery to connection points.

According to the National Institute of Statistics at the end of 2009 the length of water distribution network was 60,456.4 km, total length of sewerage network was 20,953.3 km, of which 18,367.5 km in towns and cities, and the total length of pipelines for natural gas was 33,338.4 km, of which 19,724.5 km in towns and cities.

As more complicated are localities over the terrain as underground utility lines is more complex: networks of electricity, telephone, optical fiber cables, water, sewage and gas networks. Permanently new utilities networks appear, others are relocated, there are functional networks and disposed networks.

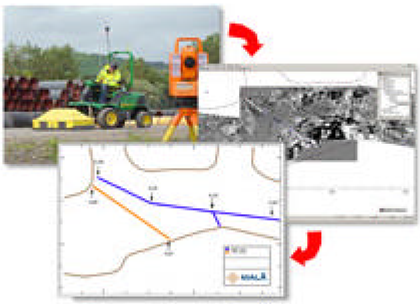

Fig. 1 Detection of the utilities and drafting plans

In Romania, according to Law 51 of 2006 and subsequent amendments and additions the regulatory authority responsible for utility services is the National Authority for Reglementation of Public Utilities Services (ANRSC). It includes water supply services, sewerage and wastewater purification, collecting, sewage and ejection of rain water, generation, transmission, distribution and supply of thermal energy in a centralized system, cities sanitation, public street lighting and management of the public and private domain of the administrative-territorial unit and local public transport. ANRSC, under powers granted by law, issues licenses, develops methodologies and framework regulations for public utility services domain within the regulatory sphere and for these services market and also monitors the way of compliance and implementation of the applicable law to such services.

But the main utility owner companies of public utilities that are responsible for knowing their location are:

- for public services of water supply and sewerage
  - S.C. APA NOVA S.A.
  - C.N. APELE ROMÂNE S.A.
- for operating services and transportation of natural gas
  - S.N.P. PETROM S.A.
  - S.C. DISTRIGAZ S.A.
  - S.N.TRANSGAZ S.A.
- for electric network services
  - S.C. ELECTRICA S.A.
  - C.N. TRANSELECTRICA S.A.
- for public lighting services
  - S.C. LUXTEN LIGHTING COMPANY S.A.
- for telephone network services
  - S.C. ROMTELECOM S.A.
  - S.C. RCS & RDS S.A.

In Europe there is a regulatory authority for the location of underground utilities using the georadar method: „European GPR Association". It provides procedures and guidelines for the design of utilities maps and plans.

Seeing the dynamics of investments in Romania the maps of utility networks are changing continuously, issue which can be controlled only by strict legal rules.

The Government Decision 28/2008 concerning of the framework of the feasibility study it is planned description of the situation of existing utilities and the need of utility networks for the proposed option, technical solutions for the utilities and costs for providing the necessary utilities for the objective, but there are not introduced any costs for detection of existing utilities.

1.3. The use of nondestructive methods in the utilities services domain

Locating the utilities and designing plans can be achieved using non-destructive techniques for:

- identifing the utilities for relocating purposes
- installation of new utility nearby communication routes
- avoiding damage of utilities during construction of new highways and national roads.

For entrepreneurs in the field of road construction work, damaging buried utilities leads to additional costs and disruption of works, which will have a negative impact to the execution period. For this reason before starting the excavations for earthworks purposes it is absolutely necessary to contract services to collate the information provided by the companies that owns utilities and those that will be obtained on field by modern detection techniques.

2. NONDESTRUCTIVE METHODS FOR UTILITIES DETECTION

The use of these methods of detection will lead to precisely locate the utilities' paths crossed by future road infrastructure , to the protection against risks related to occupational health and safety and will minimize the costs of repair for networks damaged during the excavation. Depth, position and type of utility can be determined without disturbance of the soil.

The geophysical research methods involves the use of nondestructive techniques that can detect underground anomalies without the need for excavation. The geophysical methods for utilities detection are magnetometric method and georadar method, each method having its own limitations, advantages and costs.

2.1. Magnetometric method

Through the magnetometry is measuring natural Earth's magnetic field and its variations given by the encountered underground structures (ducts, pipes, electrical cables, cavities, geology, etc.).

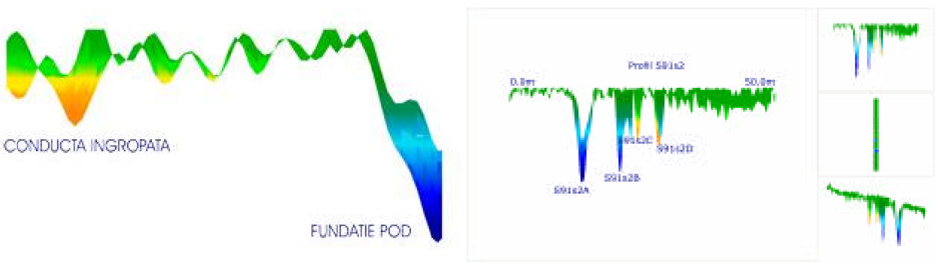

Fig. 2 Vertical magnetic sections

The images are processed in real time using a 3D application, enabling immediate on-site interpretation and further detailed analysis.

2.2. Georadar method

Geomagnetic techniques are most often non-destructive techniques widely used throughout the world in various fields. It is mainly used to locate buried utilities, unexploded munitions, archaeological sites, monitoring of environmental pollutants, etc.

Georadar method (Ground Penetrating Radar - GPR) is a nondestructive technique based on the principle of electromagnetic wave propagation in soil, rocks or any other subjects for exploration, characterization and monitoring of subsoil. This may be applied to define structures showing contrasting electromagnetic characteristics (conductivity and permittivity) against the environment. The analog to digital data acquisition of the GPR system allows rapid and quasi-continuous measurements and also rapid disclosure, in situ, of the buried structures and swift processing of obtained records. Propagating into the earth, the waves are reflected or diffracted by the interfaces which limits the structures with electromagnetic contrasting characteristics and are delivered to the surface, where are captured by another antenna, and then recorded as time function.

GPR method offers the advantage of locating metallic and non-ferrous utilities and also three-dimensional measurements.

Fig. 3Data acquisition on existing communication routes and on unbuilt land

Analysis may include a single method or a combination of several methods. Equipment must be calibrated to work in various environmental conditions and provide images in real time with minimal interruption of traffic by sending data to the storage and visualization directly on a computer screen. Personnel must be trained to properly perform in situ testing and use equipment.

One may detect any network utility position beneath the existing road infrastructure, marking their path on the road surface with biodegradable paint of different colors for different types of utilities.

Fig. 4 Marking on the site of existing utility networks

The images received are analyzed and interpreted by geophysicists who are specialists in the latest technologies and uses specialized applied software.

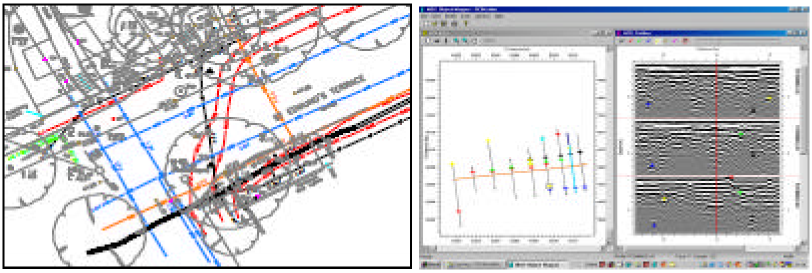

Fig. 5 The complete topographic plan of the utilities

3. CONCLUSIONS

On the entire highway network in the execution phase the cost of utilities to be relocated increases the original contract cost by significant amount. Examples of such situations are as follows:

- Transylvania Motorway
- Arad-Timisoara Motorway
- Bypass Lugoj Motorway
- Bypass Sibiu Motorway
- South Bypass of Bucharest Motorway.

The most relevant example is the Bypass Lugoj Motorway where on the 9.6 km length the utilities relocation cost was 5.5 million euros, of which gas duct Timisoara - Bucharest costs around 4 million euros.

With other words one km of highway with utility relocation costs around 2.2 million and one km of highway with no utility relocation costs around 1.6 million euros.

In 2009 the Center for Road Technical Studies and Informatics, subunit of the Romanian National Company for Motorways and National Roads acquired under the PHARE project 2006/018-147.03.10.01.04 „Improving the quality of programming and the quality of construction and rehabilitation works for bridges” a GPR to be used, at request of contractors, to detect utilities which does not exist on plans, but are present on the field, to avoid its damaging and minimize costs and execution period.

However, identifying the utilities from the route of future road infrastructure works would be appropriate if that requirement should be introduced in the framework of economic and technical documentation related to public investments regulated by Government Decision no. 28 of 9 January 2008.

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Best decision components for selection of flexible pavement ruination treatment methods

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Summary

Management of transportation infrastructure has become intricate. There are many selections of pavement preservation and rehabilitation. It is difficult for pavement engineers to select a flexible pavement ruination treatment method; too, it is totally confusing for most city council. They have to make reasonable decisions to ensure the best use of budget recourses in maintaining weakening road [1, 2]. All methods are not equivalent and many have significant restrictions and their capacity to resolve specific road distresses is fixed.

This paper describes six stags for pavement designer to select best decision components for selection of flexible pavement decay treatment methods. This study determined effective parameters by interview with municipality expert engineers and using analytical method to selecting a pavement destroyed treatment method.

KEYWORDS: Analytical method; Preventive maintenance; Flexible pavement; Crack sealing and filling, Decision component.

1. INTRODUCTION

A combination some factors deteriorate pavement. Environmental, such as aging and load related effects are the principal mechanisms of asphalt or flexible pavement decline. Structural distresses such as fatigue cracking and rutting Load related forces result. The decisions facing road authorities with respect to road treatment were simple, hot mix overlay or total reconstruction almost for fifteen years ago[1]. Then recycling and reclaiming usage techniques were increased by pavement designer and engineers. For the maintenance and preservation of existing pavements many special techniques are available and main categories can be represented into the subsequent [2]:

-Regular Preservation Treatments

1. Patching
2. Crack sealing and filling

- Preventive Preservation Treatments:
    1. Hot Mixed Asphalt Overlays
    2. Micro-Surfacing
    3. Chip Seal
    4. Fog Seals
    5. Slurry Seal

The rate of pavement deterioration is slowed by use of preventive maintenance treatments also, thereby the need major rehabilitation delaying by several years.

This paper illustrates, in detail, the methodology used by analytical method to collect and analyze the data for selection of flexible pavement ruination treatment methods [3]. The results of this analysis are then explored in the form of budget scenarios and funding recommendations.

Best decision components for selection of flexible pavement ruination treatment methods by analytical evaluation procedure included six main stages:
Stage 1: Recognize the pavement system;
Stage 2: Determine pavement management segments;
Stage 3: Survey pavement present condition;
Stage 4: Use software for computation strains in pavement layer in different depths;
Stage 5: Calculate extended pavement life and pavement maintenance life cycle cost, with noticing viewpoint of municipality expert engineers;
Stage 6: Comparison and Select best pavement maintenance method based on minimum life cycle cost; in this stage choosing matrix is created by pavement maintenance manager and designer.

2. LITERATURE REVIEW

Duangi and his research team used four parameters for assessment of pavement condition of an expressway in Guangzhou, as [4]:
1-PCI, Pavement Condition Index
2-RQI, Riding Quality Index
3-PSSI, Pavement Structure Strengths Index
4-SRI, Slip Resistance Index

In above research done a multivariate analysis of PCI and appeared a correlation between the two survey pavement condition indexes.
Research work done by Morosiuk and his colleague studied pavement maintenance choosing methods by using HDM-4 software, purpose of this

implicated research was improve of mix design procedure, specifications, construction practice and structure design [5].
A comparison between pavement maintenance selecting methods about economic parameter done by Hein and his cooperator, that showed pavement preventive maintenance is economic than overlay hot mixed asphalt [6].A paper presented by Zimmerman and his colleague discussed integration between pavement management system and pavement preventive maintenance. In this relation, data that collected by PMS can be used in pavement preventive maintenance method selection [7].

3. RESULTS OF EXPERT ENGINEERS QUERY

For determining best decision components by pavement expert and designer completed about 15 questioners' forms. Table 1 shows results of this surveying. Attention to results illustrated that economic factor and treatment life have best situation between decision components.

Table 1. Results of expert engineers query about best decision components

Parameters	value quantify				Percents%
	High	**Medium**	**low**	**Very low**	
Economic factor	**11**	**2**	**1**	**0**	73
Treatment life	**9**	**4**	**2**	**0**	60
Equipment	**5**	**5**	**4**	**1**	33
Practice experiment	**5**	**4**	**3**	**3**	33
Negative impact environment	**4**	**4**	**5**	**2**	26

Therefore value matrix in AHP method for selection pavement treatment can be present as followed matrix [8]:

$$\text{Value Matrix}=\begin{bmatrix} 73 \\ 60 \end{bmatrix}_{2\times 1} \tag{1}$$

4. USE OF ANALYTICAL METHOD

To compute stress and strain values of pavements at different locations during the depth of the pavement structure employed Kenlayer software. This computer software was used to calculate two strains [9, 10]:

- The tensile strain at the bottom of the flexible layer; and
- The compressive strain at the top of the granular subgrade.

These computed strains are used in the rutting and fatigue cracking model to calculate the different pavement maintenance life.

4.1. Stress and Strain Analysis

Different elasticity modules of pavement maintenance are measured in this research; the standard axle load is 8.2 tons. The assumed axles are the most public; ones contain two sets of double tires with 5.6kg/cm^2 contact tire pressure and 45 cm double spacing [9, 10].

To determine the increasing quantity of pavement lifetime, it has made use of two Equations (2) and (3) which are presented by Shell petroleum research office [10, 11]:

$$N_r = \left(\frac{0.0028}{e_v}\right)^4 \tag{2}$$

$$N_c = 0.0685\left(\frac{1}{e_t}\right)^{5.671} \times (E_1)^{-2.363} \tag{3}$$

Where,

N_r = the number of equivalent standard axles to final pavement serviceability index of 2.5.

e_v =vertical compressive at the top of the subgrade,

N_c = number of load repetition to the failure by fatigue cracking,

e_t = horizontal tensile strains at the bottom of the asphalt layer,

E_1 = asphalt elasticity modulus.

4.2. Analysis Results

In this paper tree pavement treatment methods select for assessment, operation cost and their life took from previous research results [12, 13].these methods included as:

-Micro-Surfacing (10% increasing in elasticity module of surface asphalt layer);

-Slurry Seal, (7% increasing in elasticity module of surface asphalt layer), and

-Crack filling and sealing, (5% increasing in elasticity module of surface asphalt layer).

By considering increasing elasticity module of surface asphalt layer in Kenlayer software, the tensile strain at the bottom of the flexible layer and the compressive strain at the top of the subgrade decreased and pavement's life increased. If number of load repetition until a generated failure calculated by above equations, Micro-Surfacing, Slurry Seal and Crack filling and sealing increase pavement life about 2.1, 1.3 and 1.1 consecutively. Therefore AHP Matrix [8] created as Equation (4):

$$\text{Choosing Matrix}=\begin{bmatrix} 2 & 2.5 & 4 \\ 2.1 & 1.3 & 1.1 \end{bmatrix}_{3\times 2} \times \begin{bmatrix} 73 \\ 60 \end{bmatrix}_{2\times 1} = \begin{bmatrix} 270 & 260 & 358 \end{bmatrix} \quad (1)$$

By attention to calculation of choosing matrix, best pavement maintenance is Crack filling and sealing.

5. CONCLUSIONS

The paper has explained a methodical approach to investigate the appropriate maintenance strategy for the pavement management system by using Analytical method with pavement strains under load effect. From this analysis, the following can be concluded:

1. Results of expert engineers query introduced best decision component respect is followed:
   - Economic parameters and pavement treatment cost decreasing,
   - Extended pavement life,

Therefore, first parameter that had to be attention is treatment cost.

2. With attend to analytical results, if surface layer elasticity module increases about 10%, then pavement life increase about 2.1 times.
3. Crack filling and sealing pavement treatment method is best choosing decision in this study, because this method had minimum cost operation and maximum preventive life.
4. This study can be followed by researching about filed data collection by FWD machine test in the future, and those results compare with analytical method (this study outcome).

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Life cycle assessment application for highway pavements environmental impact

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Summary

This paper is a study on Life Cycle Assessment methodology and its application for the evaluation of environmental impact of highway pavements asphalts mixtures. The evaluation is based on the TRL software, asPECT which can be used for the determination of carbon dioxide equivalent emissions CO_2e for a specific asphalt mixture. The example uses BAD 25 asphalt mixture, the percentages used in the composition have been taken from Ecolanes Project but the rest of the information in the program are fictive.

The results wish to highlight the impact of different asphalt mixtures used in highway pavements to the environment. This type of analysis can be used in order to compare the degree of environmental impact of different mixtures.

KEYWORDS: life cycle assessment, environmental impact, asPECT software, asphalt mixture.

1. INTRODUCTION

In the context of a generalized application of sustainable development also for the construction sector, an increased population growth, a higher and more complex demand for transport infrastructure as well as raw materials scarcity comes into sight the ecological impact of all these changes.

This way is important to evaluate the environmental burden associated with any engineering project in order to take the right decision from several points of view like economic, social and environmental.

The environmental impact associated with different asphalt mixtures used for highway pavements is necessary to be evaluated first of all for a proper selection of the materials. In order for companies to improve the environmental performance are needed new evaluation instruments and methodologies as well as computer software.

Life Cycle Assessment (LCA) is an instrument which evaluates the environmental impact associated with products, procedures and activities. Although controversial and complex, the instrument had a continuous evolution since its introduction and its applicability extended in different areas of research. The LCA instrument brought to light several other instruments like LCCA (Life Cycle Cost Analysis), LCI (Life Cycle Inventory), etc. In the last years this methodology has been introduced in different computer software in order to facilitate the products or processes evaluation. [1]

The TRL software, asPECT uses LCA for the evaluation of environmental impact of asphalt mixtures for highway pavements. Currently, the software presented a new version, with an improved interface and complexity.[2]

2. LIFE CYCLE ASSESSMENT METHODOLOGY

LCA has been defined by the standard ISO 14044 as the methodology for the evaluation of inputs and outputs and of the potential environmental impacts of a product system during its lifetime. By inputs are considered the required resources for the process and by outputs the emissions to different compartments as air, water or soil. [1]

LCA methodology came to life around 1960 due to the fast exhaustion of fossil fuel and it was used like an approach in order to understand the impact of energy consumption. [4] In this context, LCA became a system approach for the evaluation of environmental consequences for a product, process or activity from “cradle to grave”. It observes the whole life cycle of a product from raw materials extraction and materials production till final disposal. The life cycle for a product can be seen in fig. 1.

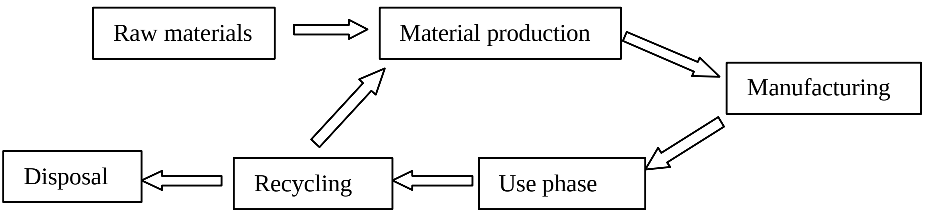

Figure. 1 Life Cycle of a product [1]

By using LCA can be identified the environmental hotspots [1] of a product or process and use this information for the improvement of environmental performance at every stage of a product life cycle.

The direct applications of a LCA are:

- Product development and improvement,
- Strategic planning,

- Public policy making,
- Marketing,
- Other.

A LCA framework contains among other elements the impact assessment. The Life Cycle Impact Assessment (LCIA) evaluates the environmental impact of a product system by so called impact categories. These categories are definitions of which substances contributed to a certain environmental problem, for example CO_2e carbon dioxide equivalent emissions is the reference substance for global worming impact category. The substances can contribute to more than one impact category.

The environmental impact includes the emissions added to the environment from each life cycle process of a material. This is the reason for which currently get more and more attention the materials selection process but also the creation of new materials so called composite materials. Majority of these composites have the advantage of being sustainable materials. They are environmentally friendly, less expensive, easier to use in a construction work and more efficiently.

3. ASPECT SOFTWARE DETAILS AND APPLICATION

The extent to which LCA is being used, created the necessity of facilitation the work by introducing the data in different computer software. Currently, in the construction area is being used more than one program of this type. We can mention here GaBi 4, LCCOST, SimaPro, RealCost, AsPECT, etc.

The asPECT (asphalt Pavement Embodied Carbon Tool) software was created alongside the *Protocol for the Calculation of Life Cycle Greenhouse Gases Generated by Asphalt used in Highways* and following in the paper can be found some information about the program characteristics and data from an example of recycling materials used in a construction work.

The asPECT software enables the user to calculate the CO2e emissions associated with certain asphalt products. The software gathers information on the materials used, transportation and mixing plant characteristics in order to evaluate the product and the CO2e emissions generated in order to compute the values per ton for each life cycle step and mixture in the project.

Currently asPECT software has two versions: version Alfa which is a computer program based on Microsoft Excel and a Beta version starting with October 2010 with a new interface and without using Microsoft Excel. In figure 2 can be seen both versions of the software opening page.

Fig. 2.1 Alfa version of asPECT software

Fig. 2.2 Beta version of asPECT software

The user, while using the program must complete a series of forms in order to evaluate the CO2e emissions associated with asphalt products. The information entered by the user together with the emissions factors and default data referred to in the protocol are being used to conduct the assessments.

Both versions of the software follow the same structure: materials, plants and projects. The entry forms guide the user through a partial life cycle CO_2e emissions assessment for asphalt, using asphalt plants as a focus.

The first few forms are concerned with building a “materials” database, where is captured information relating to raw material acquisition. The mixture applied in the example is based on a composition used in Ecolanes project named BAD 25 which contains seven raw materials, six of them being newly introduced materials in the software database. The process to create each material implies following four steps. Can be entered as many new materials as someone may need as long as are followed the four mandatory steps. [2]

The materials and their percentage found in the BAD 25 mixture used for the example are bitumen 6%, chipping size 16-25, 21%, chipping size 8-16, 21%, chipping size 4-8, 8%, natural sand 15%, crushed sand 20%, hydrated lime 9%. [5]

Second stage “Add plant data” is concerned with the characteristics of plants used to batch asphalt. Complementary are added transport information. The products can be mixed from different plants in the required quantities but is also necessary to add laying and compacting data in order to make an assessment at this level. This phase is a succession of 6 steps in which is being created a plant with the materials and characteristics imposed by the user. The plant used in the example is a “recycle plant”.[2]

The final phase in the main menu is to “Create a project”. Once the recycle plant has been adequately defined the whole project can be modeled. The “Summary Results” screen, gives the total and per ton inputs for material and transport along with the kg CO2e produced in each life cycle step (Fig. 3).

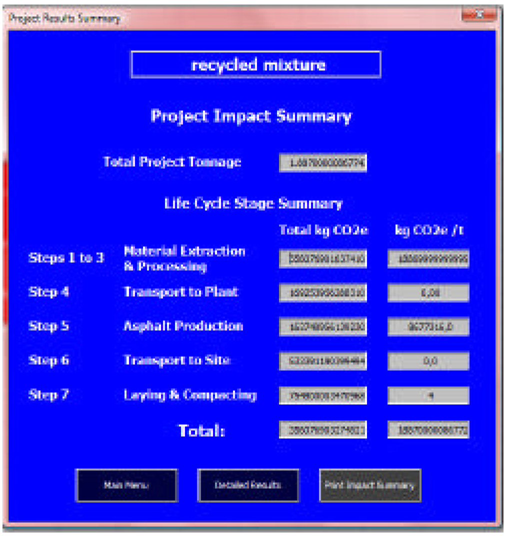

Fig. 3 Project Impact Summary

The software has the facility to print a PDF versions of the analysis results. Figure 4 contains the example results.

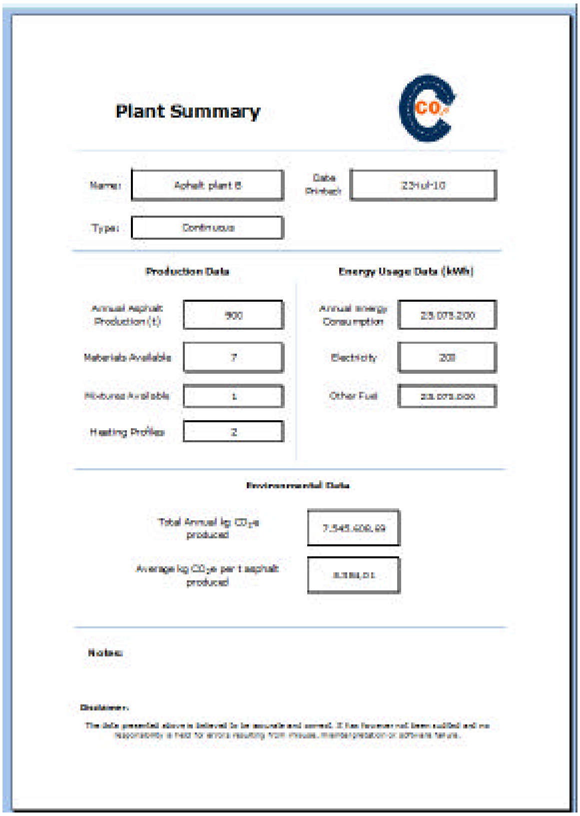

Fig. 4 Project results in PDF version

4. CONCLUSIONS

The paper highlights the need for higher consideration of environmental parameters in decision making in the transportation area and that LCA is an optimal methodology to realize such an evaluation.

The results obtained by using the asPECT software, even though the majority of the information is fictive, allows us to see the utility of the computer software and of LCA analysis in the evaluation of environmental impact of an asphalt mixture.

A transportation agency can use this computer program for the selection of an optimal mixture for roads pavements from environmental point of view.

Currently this software or other similar to it find themselves a great applicability in the transportation area and the agencies have the duty to use them for the future projects decisions.

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Ireland’s Transport Infrastructure

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Summary

This paper presents the transport infrastructure of Ireland. In the first part of this paper is presented a short introduction regarding the economic evolution of Ireland and the necessity of transport development in this country. In the following part are considered: the importance of transportation in Ireland that is affecting everything from the day-to-day lives of common people to the economic success of the nation as a whole, a short presentation of the Waterways, Airways, Railways, and Roadways. Finally are presented the roads category used in Ireland and a short analyze between the percent of all roads which each type of category constitutes and the actual percent of traffic which each carries.

KEYWORDS: transport infrastructure, road network, roads category

1. INTRODUCTION

Ireland is the third-largest island in Europe and the twentieth-largest island in the world, with an area of only 84,288 km^2, a maximum length of 485 km, and a maximum width of 304 km [1]. It lies to the northwest of continental Europe and is surrounded by hundreds of islands and islets.

The population of Ireland is approximately 6.2 million people. Just fewer than 4.5 million live in the Republic of Ireland and just fewer than 1.8 million live in Northern Ireland.

Ireland traditionally had a low investment in its roads infrastructure principally due to a continuing lack of available funds for investment in the period up to the 1990s [2]. In the last 10/15 years the Irish economy has grown very quickly and the demand this has placed on all public utilities led to serious concern that such lack of investment would hamper the sustainability of this economic growth.

There is an extensive road network and a developing motorway network fanning out from Dublin and Belfast in particular. In recent years, the Irish Government launched a new transport plan that is the largest investment project in Ireland's transport system: investing €34 billion from 2006 until 2015. Work on a number of road projects has already commenced and a number of objectives have been completed [3].

2. THE IMPORTANCE OF TRANSPORTATION IN IRELAND

Safe and efficient transportation of people and goods is one of the biggest challenges that Ireland faces today. The importance of a transportation network is far-reaching, affecting everything from the day-to-day lives of common people to the economic success of the nation as a whole. People desire the most cost-effective and efficient modes of transportation for both business and personal travel.

Transportation is even more critical in the commercial world. Ireland is highly dependent on revenues from exported goods, which must leave the country and arrive at their destination unscathed and on time. Within the nation, population and economic development are widely dispersed, and companies therefore rely heavily on the transportation network to move both goods and workers [4].

One of the keys to Ireland’s continued economic success has been its production of high-value-added products which are transported in relatively low volumes and must be moved to and from a wide variety of places with the most speed possible.

The substantial dependence on agriculture is another reason for the importance of a high quality transportation network. Tourism, another important economic sector, requires an extensive transportation network so that visitors may frequent the rustic towns, ancient castles, and spectacular cliffs of the less populated regions as well as the many attractions in Dublin and the other cities [4].

2.1 Ireland’s Transportation : Waterways, Airways, Railways, and Roadways

In Ireland, transportation options include sea, air, rail, and road travel. As an island nation, Ireland relies exclusively on air and sea travel for transportation to and from other countries. There are 44 airports, [5], the two largest international airports, one on the west coast and one on the east, are in Shannon and Dublin respectively. There are also nine ports and harbors and 700 km of waterways in the country [5]. These modes of transportation are vital for external travel and exporting and therefore have been improved and expanded as the economy has dictated.

Air and sea travel within Ireland, is limited by both cost and accessibility issues. In regards to sea travel, although there are inland waterways, all of the major ports and harbors lie along Ireland's shores and therefore provide little access to inland destinations. Sea shipping is used mainly to transport lower value, high-volume cargo.

Since air and sea travel are not viable options within Ireland, internal transportation relies on railways and to a much greater degree on roadways. There are almost 2,000 km of railways [5] as well as a light rail system (the DART) in the greater Dublin area. Both the main rail and the light rail system are excellent alternatives to road transportation for goods and people. They are limited, however, in their usage because of capacity and extent.

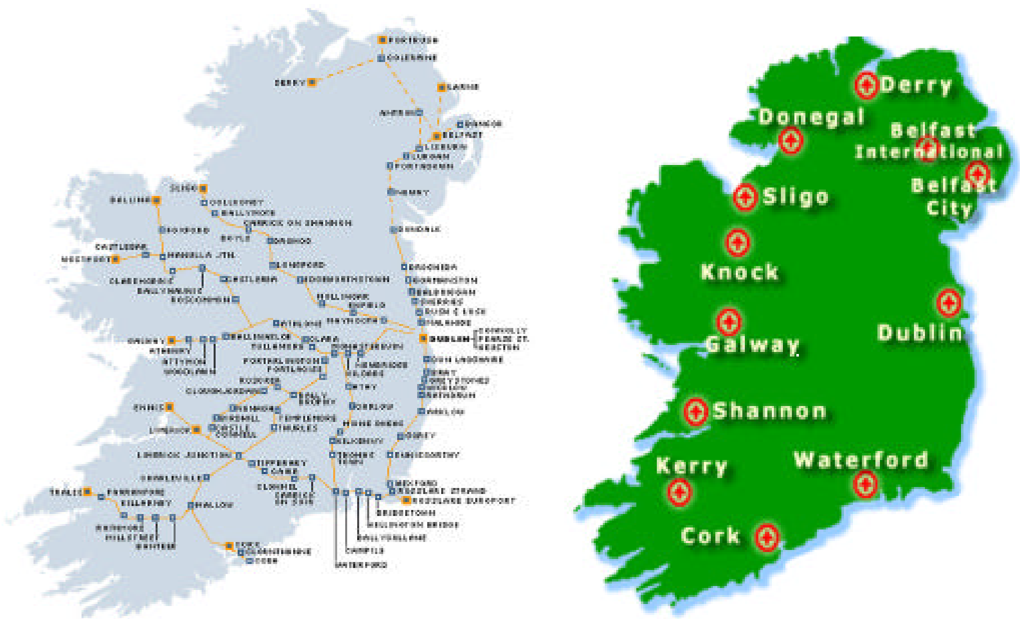

Figure 1. Ireland's railway and airway networks

The main component of transportation in Ireland is the road network. Ireland, more than other European Union countries, relies almost exclusively on its roadways for inland passenger transport.

A total of 97 percent (91 percent by car, six percent by bus) of passenger transportation is by road, while the remaining three percent takes place via railway [6].

3. ROADS IN IRELAND

There are five main road classifications: motorways, national primary, national secondary, regional, and local. In Ireland, the highest category of road is a motorway, the national primary roads are the major long-distance through-routes, which link principal population centers and thus serve a high percentage of the population. The national secondary roads are medium-distance through-routes connecting smaller key towns with links to the primary routes. Regional roads act as the main links between the national roads. Finally, local roads consist of all rural and urban roads that are not included in one of the other categories [4].

The Republic has an extensive network of public roads connecting all parts of the country. As of 31 December 2009, there were a total of 5 443,726 km of national roads: 2 735.800 km of national primary routes (including motorways) and 2 707.926 km of national secondary routes [7]. In addition to national roads, the Republic has also an extensive network of other public roads: there are 11,630 kilometers of regional roads and 78,972 kilometers of local roads [8].

Figure 2. Ireland’s roadway network

Figure 3. Road junction on the M3 motorway Dublin

In figure 3 is presented an example of Ireland's motorway.

Figure 4, below, shows the stark disparity between the percent of all roads which each type of category constitutes and the actual percent of traffic which each carries. While comprising only three percent of the total road network, primary roads carry 27 percent of total road traffic. National secondary roads comprise another three percent of the road network while carrying eleven percent of total road traffic. Regional roads make up eleven percent of the road network and carry 24 percent of the road traffic. The local roads make up 83 percent of the road network but carry only 38 percent of the road traffic [4].

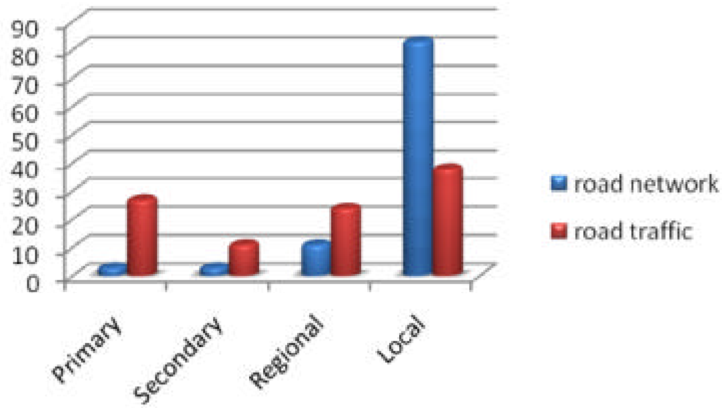

Figure 4. Percentage of each category of roads and the traffic which each carries

4. CONCLUSIONS

Considering the great importance of a transportation network in Ireland and the key role of roadways in this network, it should come as no surprise that Ireland has invested heavily in its entire transportation network and particularly in its roadways. At present, there are over 350 km of new motorway under construction, due to be completed by the end of 2010 or earlier. Recent redesignation of dual-carriageway sections of National Primary routes mean that there will be approximately 1000 km of motorway in the Republic of Ireland by the end of 2010 with further planned construction possibly leading to around 1200 km of motorway by 2015.

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Some legal aspects in civil engineering activities

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Abstract.

As a rule, the development level of a country is determined by making reference to her heritage or the construction extent of the corresponding country. The better is legally organized the private real estate field and the more extended is the judicial protection granted to the real estate, the higher is the interest in the development of the real estate promotion.

The present work proposes an analysis of some legal aspects governing the civil engineering field/sector in Romania, more exactly, the judicial situations resulted from the development of the building activities, which are related to land choice, the contracts concluded between parties, authorizations, the quality of the utilized materials, the works organization on the yard, the taxation problems and least but not last, the financing sources.

Key words: law, civil engineering, real estate, land, contracts.

1. INTRODUCTION

The period after 1990 has known an unpreceeding development in civil engineering field, even if not without difficulties. Irrespective of the fact that family dweelings, flats complexes, buildings for offices, emporiums or businesses were built- all the people involved have met a series of bureaucratic obstacles. The quality of the works in construction is also a problem for the real estate investors.

The obstacles we were talking about concern choosing the land, contracts concluded between the parties, the quality of the utilized materials, the works organization on the yard, taxation problems, financing sources related to: granting the land, renting the buildings, establishing of real rights of superficies or servitutes, bordering, real-estate leasing, registration of the ownership or of other rights in terms of land and buildings, according to the legal stipulations.

2. CHOOSING THE LAND

The first step in the realization of a construction is to choose the land on which the building is to be located.

There are multiple and diverse offers related to lands, mainly because nowadays, given the financial situation of most of the citizens, those willing and having at their disposal financial resources for such investments, have the possibility to choose lands at prices unthinkable two- three years ago. Even though, most of those willing to purchase a land put on the map the financial aspect of the real estate business, and less the judicial effects resulting from the corresponding transaction.

When we speak about the judicial aspects deriving from such an operation, we refer to: checking up the title deed on the land which make the object of the transaction, the real surface, neighbourings, land configuration in terms of the construction to be built, etc.

From the moment the land is acquired, begins the project development, in which participate architects, designing engineers and builders.

3. CONTRACTS BETWEEN THE PARTIES INVOLVED

The Contracts concluded by the beneficiaries with the architects, designing and building engineers and often result in missunderstandings that can only be solved by law. The judicial costs resulting from such missunderstandings are quite big, to which we must add wasting a very precious time in lawsuits and not lastly- loosing money.

To all these, one must add the discommode generated to the direct and indirect beneficiaries of these contracts. In order to avoid all these, people started to estimate otherwise their interests in conflicting situations, since it is more important to win than to be declared vanquished.

A rigorous calculation of the cost of a lawsuit and the analysis of risks and time wasted in the law court can constitute enough reasons to prefer the alternative variants of the classical justice.

In Occident, looking for the means to solve the conflicts on other than judicial ways, courts and other instances, represented a constant concern even since the 70's. In Romania, an alternative to the classical solution was found late and it is represented by the mediation institution, regulated by the Law No. 192/2006.

In fact, what is the mediation? The mediation represents an optional modality to solve the conflict in an amiable manner, by means of a third specialized party in capacity of mediator, under the conditions of neutrality, impartiality and confidentiality.

According to art.1, items 1 and 2 of the Law 192/2006, the mediation is based on the trust the both parties grant to the mediator, as a person able to facilitate the negociations between them and to support them to solve the conflict, by producing a win-win efficient and durable solution. In this connection, the Law 192/2006 stipulates at art. 2 items 1 and 3, that the natural or the legal persons have the right to solve their disputes by mediation both outside and inside the mandatory procedures, for an amiable solution of conflicts stipulated by the laws.

4. BUILDING LICENSE

The relations with the administrative authorities represent a very important segment/component in the lawsuit of a building or a real estate complex. These make reference to: getting the planning certificate, the building and environment authorizations, the special authorizations or notices in case of a construction with special destination, etc. It is well known that there are several obstacles met by the investment beneficiary in order to obtain these authorizations and notices, without which the construction can not begin.

The building license is the deed of authority of the local public administration on whose basis the construction works can be executed and on whose bases the application of the legal measures concerning the location, design, execution and operation of the objectives of an investment are ensured. At the same time, one has to know that there are ways of attack which the injured persons can use against the authorities who refuse, with or without a reason, to release the construction authorizations and advices.

5. THE QUALITY OF THE MATERIALS UTILIZED IN THE BUILDING REALIZATION

The utilized building materials can be purchased without difficulties, both from the country and abroad. Yet, it is important to keep in mind that the materials- more exactly their quality- are appreciated according to certain parameters specific for civil engineering.

Non-compliance with these quality requirements can entail the supplier accountability to the extent to which the beneficiary or the builder took an interest in this thing. In connection with this aspect of the quality of construction materials, a series of norms/normatives were issued, whose non-compliance will entail consequences for the party who had by contract the obligation to purchase them. In this connection, in the matter of a contract for works, the Civil Code makes a distinction between two possible cases, depending on the fact that the materials the construction is build of belong to the builder or to the customer (the beneficiary).

According to art.1479 of the Civil Code, if the materials were purchased by the builder, who is also the owner of the specified goods/things, on reception he will stand the risk of fortuitously loosing them.

Yet, if the customer was set in delay (was notified viva voce or in writting) by the builder in view of work reception, and the customer did not proceed to taking them over, than the risks are incumbent to him.

In the situation when the materials belong to the beneficiary, as the owner of the materials, he takes the risk of loosing the goods/things. Yet, there is an exception from this rule, namely that, even if the materials belonged to the beneficiary, but the builder did not kept them under adequate conditions, such that their degradation resulted in the perish of the good/thing. Under this situation, the builder is culpable for good/thing perishing. The Civil Code specifies at art. 1481: „if the thing perishes due to a vice of the materials purchased by the customer, the builder has the right to pretend the cost payment”. In this case, in fact it is not about the risks (since the destruction did not occur from a fortuitous or majeure force case), but about standing the damage caused by the customer having purchased inadequate materials.

6. WORK ORGANIZATION

The organization of a building yard is a complex activity, which involves a high degree of responsibility. Each person involved in the activity of a building yard must have well established attributions, and complying with these attributions must be checked by the persons abilitated to carry out such activities. All these need to be stipulated in the contracts or subcontracts of works execution concluded with the beneficiary, as well as in the contracts which the general builder or subcontractors concluded with the doers.

Most of the time, the content of these contracts includes certain minimal clauses which, should they be known, would result in avoiding the risk of generating conflicting situations from the part of both the beneficiary and the builder.

The accountability in the real estate matter generally raises a series of problems, for which reason we consider mandatory to be aware of the effects of assuming various forms of this liabilities (civil liability, penal liability, contravention liability, etc).

7. FINANCIAL SOURCES

The financial resources which represent the basic element for the realization of a real estate project are determined in terms of size and destination of a building. The banks, the real estate companies, the natural persons, each of them can provide the corresponding financing sources.

Choosing the financing sources implies a series of previous checking on the real landing or association capability to finance a project. Such checking is also imposed by the fact that the crysis situation that affected Romania during the last two years seems not to end too soon, and the lending conditions of the financial institutions are not quite accessible or attractive, existing the risk that the real estate project can not finished.

The guarranties constituted in order to obtain the long or middle term credits, such as mortgages or personal guarranties, impose the knowledge of some legal aspects specific to the constitution and execution of these guarranties.

8. RISK INSURANCE

It is well known that the risk insurance in the real estate field raises problems. The situation is similar in the case of the architect and designing engineer professional liability insurance, real estate insurance in case of damages produced by earthquakes, blaze, floods or ather similar situations, as well as in the case of insurance for damages caused by the builder in carrying out with the obligations he assumed toward the beneficiary.

The real estate exploitation generates several other judicial situations, such as: land concession, renting the buildings, constitution of real rights, such as superficies and servitutes, real estate leasing, registration of ownership title or of other rights concerning the building lands, according to the present legal stipulations, etc.

For the documentation and getting the information related to the real estate field, one resorts to various categories of specialists, for example: architects, designers, builders, designing engineers, suppliers. Within the activities carried out by the above mentioned specialists, a series of special rights can be generated, which imply a special protection, more exactly the copyrights for the project/work, in the case of the architects and designers, or the logo used by a real estate company/society for the provided products and services.

For the real estate proffesionals and specialists, the judicial information/knowledge are indispensable in order to carry out their activity; that is why it is mandatory for a real estate project to be advised by a law specialist, as a rule by a lawer. A first advantage presented by such an advice is that the price of the counceling diminishes the judicial risks that might appear in case of a real estate project in the absence of minimum recommendations coming from a law specialist.

As for the legal regulations in force in the real estate matter, we consider that at this moment there are in Romania law norms which cover the real estate field, for example the Civil Code which

contains the general principles, as well as a series of special laws which come to make it complete.

In turn, the special laws which regulate the real estate field contain stipulations related to a multitude of situations that can appear during the real estate project realization. These situations make reference to professional liability of the architect of the designer, the responsibility for the building quality, stipulated at the dispositions of art. 1483 of the Civil Code and those of art. 29 of the Law No.10/1995 concerning the quality in construction.

At the same time, other situations that can come across in the realization of a real estate project are those concerning the insurances, environmental protection special real estate contracts, concessions of real estate or real estate goods and services, etc.

9. CONCLUSIONS

The development of the real estate field, especially of that of constructions, can not develop outside the corresponding legislation.

A correct and complete information in such a complex field as that of civil engineering can not exist without information with judicial character. That is why it is mandatory that, besides various categories of specialists who take part in the real estate development, namely architects, designing engineers, builders, designer, suppliers of materials and real estate complexes, each real estate project is adviced by a law specialist. At the same time, it is mandatory that all these specialists carry out their activity based on the legislation that regulate this field.

Several regulative deeds were issued after 1990, stipulating the rights, the obligations and the sanctions applicable in case of non-compliance with them.

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The energy dissipated inside anti-seismic systems consisting of neoprene bearings, intended for dynamic isolation

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Summary

This paper presents the results of tests carried out upon neoprene isolators basing on both kinematic and dynamic excitation. Also, it presents the method provided by the standard using a kinematic excitation allowing the determination of viscous damping.

KEYWORDS: dynamic excitation, kinematic excitation, dissipated energy.

1. INTRODUCTION

The provisions of the European Standard SR EN 15129 specifies the repeatable technical conditions that should be applied to the testing of neoprene anti-seismic isolators. Aiming this, the quasistatic testing method is used at low velocity (periods of about 2-3 minutes) or oligocyclic with frequencies in the range (0,5 ... 5,0) Hz.

The applied exterior action represents a kinematic excitation under the form $x = A_0 \sin wt$ and the dynamic response corresponds to the linear viscoelastic system represented by the instantaneous force $Q(t) = kx(t) + c\dot{x}(t)$, where k is the stiffness and c the viscous damping.

Basing on this, the hysteresis loop has been plotted and the parameters k and c have been determined.

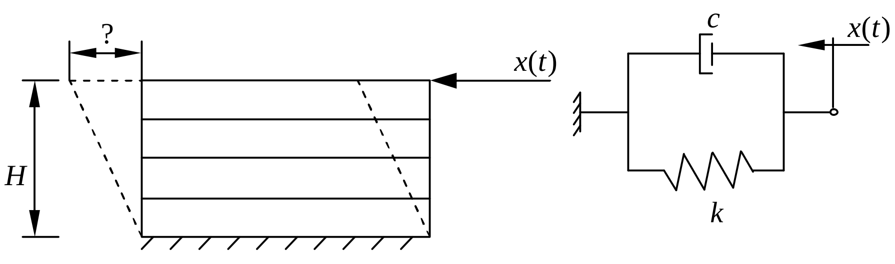
This study puts into evidence that the testing of neoprene isolators subjected to dynamic exterior actions, the hysteresis loop area changes, so that for the same isolator different results for the viscous damping could be obtained.

2. TESTING METHODS DEPENDING ON THE ACTION NATURE

2.1. Kinematic excitation

Neoprene isolators are provided with interior metallic shims and rubber layers subjected to shear loading. The shear loading prevails upon the compression one, assuring a lower stiffness and a higher damping during the seismic shock isolation process.

For the model illustrated in figure 1, the viscoelastic linear system massless subjected to exterior kinematic shear loading with the instantaneous displacement $x = A_0 \sin wt$ is put into evidence.



a. b.

Figure.1

The instantaneous viscoelastic force is given by the relation:

$$Q(t) = kx(t) + c\dot{x}(t) \tag{1}$$

Where introducing x and $\dot{x}$ we have:

$$Q(t) = Q = kx \pm cwA_0\sqrt{1 - \frac{x^2}{A_0^2}} \tag{2}$$

Represented in the axis system *Q*-*x* as an ellipse having its eigen axis inclined in respect to the axis system *Q*-*x*.

The equivalent critical damping z_e can be determined as:

$$z_e = \frac{\Delta W^c}{2pkA_0^2} \tag{3}$$

where $\Delta W^c = pcwA_0^2$ is the loop area, meaning that the energy dissipated over the cycle for applied kinematic loading; $kA_0^2 = 2W_{el}$ is the elastic energy.

One can notice that the dissipated energy ΔW^c, for given kinematic actions, has a linear dependence on *c* and ? and parabolic dependence on A_0^2.

In this case, the average dissipated power related to one cycle

$$P_c = \frac{1}{2} c w^2 A_0^2 \tag{4}$$

is represented in figure 2.

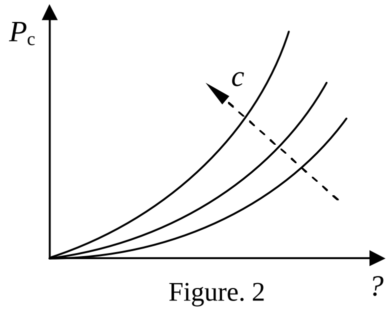


Figure. 2

2.2. Dynamic excitation

In case of dynamic excitation $F(t) = F_0 \sin wt$, with the perturbing force applied from exterior, the instantaneous displacement response is under the form $x = A(w)\sin(wt - j)$.

The applied dynamic action can be:

a) $F(t) = F_0 \sin wt$

where F_0 = const. is the amplitude of the excitation force having a consistent value for the whole test

b) $F(t) = m_0 r w^2 \sin wt$

where $m_0 r w^2$ is the amplitude of the excitation force depends on ?. The excitation is an inertial one, being generated by the rotation of the unbalanced mass m_0 located at the distance *r* in respect to the rotation axis.

The dynamic models of the massless viscoelastic system are given in figure 3.

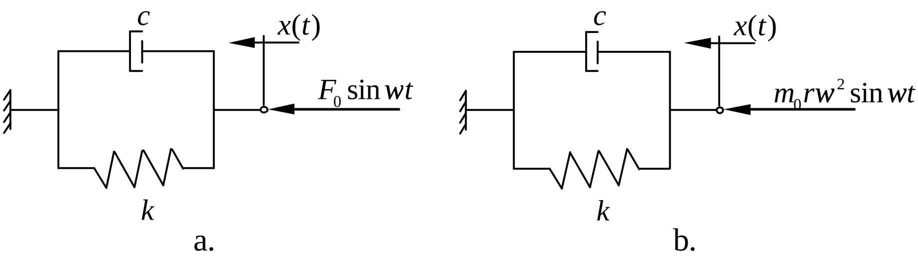


Figure. 3

2.2.a Dynamic excitation under the form $F = F_0 \sin wt$

For the system illustrated in figure 3a, we have

$$Q = kx + c\dot{x} = F_0 \sin wt \tag{5}$$

with the solution

$$x = A_1 \sin(wt - j_1) \tag{6}$$

where

A_1 is the displacement amplitude

j_1 - phase difference between force and displacement

Thus, from (5) and (6) we obtain:

$$A_1 = \frac{F_0}{k}\frac{1}{\sqrt{1+\frac{c^2w^2}{k^2}}} = \frac{F_0}{k}\frac{1}{\sqrt{1+n^2}} \tag{7}$$

$$tg j_1 = \frac{cw}{k} = n \tag{8}$$

In this case, the hysteresis loop equation is:

$$\frac{Q^2}{F_0^2} - 2\frac{Q}{F_0}\frac{x}{A_1}\frac{1}{\sqrt{1+n^2}} + \frac{x^2}{A_1^2} = \frac{n^2}{1+n^2} \tag{9}$$

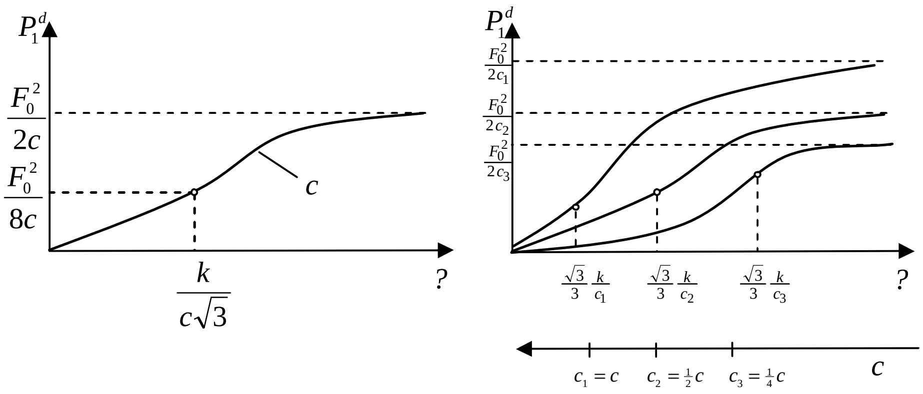
The dissipated energy ΔW_1^d depends on $A_1 = A_1(w) = A_1(n)$, so that we have

$$\Delta W_1^d = pn \frac{F_0}{k} \frac{F_0}{1+n^2} \tag{10}$$

The dissipated power P_1^d, for one cycle is given by the relation:

$$P_1^d = \frac{cF_0^2}{2} \frac{w^2}{k^2 + c^2 w^2} \tag{11}$$

with its variation presented in figure 4.



a. b.

Figure. 4

2.2.b Dynamic excitation under the form $F = m_0 r w^2 \sin wt$

From the figure 3b, one can write the following equation:

$$Q = kx + c\dot{x} = m_0 r w^2 \sin wt \tag{12}$$

with

$$x = A_2 \sin(wt - j_2) \tag{13}$$

so that, finally, one obtains:

$$A_2 = \frac{m_0 r w^2}{k} \frac{1}{\sqrt{1+n^2}} \tag{14}$$

$$tg\, j_2 = \frac{cw}{k} = n \tag{15}$$

The hysteresis loop equation is:

$$\frac{Q^2}{\left(m_0 r w^2\right)^2} - 2\frac{Q}{m_0 r w^2}\frac{x}{A_2}\frac{1}{\sqrt{1+n^2}} + \frac{x^2}{A_2^2} = \frac{n^2}{1+n^2} \qquad (16)$$

The dissipated energy ΔW_2^d depends on the amplitude $A_2 = A_2(n)$:

$$\Delta W_2^d = pn\frac{\left(m_0 r w^2\right)^2}{k\left(1+n^2\right)} \qquad (17)$$

The dissipated power is given by the relation:

$$P_2^d = \frac{1}{2}c\left(m_0 r\right)^2 \frac{w^6}{k^2 + c^2 w^2} \qquad (18)$$

having the representation in figure 5.

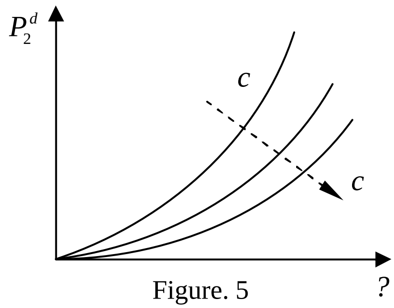


Figure. 5

3. CONCLUSIONS

For the same neoprene element, both kinematic and dynamic excited, the dissipated energy and the dissipated power, respectively are quantities with significant differences.

This is the reason why, for the final intended use of the isolator, the most suitable method, kinematic or dynamic, should be adopted.

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Iasi, România, December 10, 2010

Preliminary study of different methodologies for traffic assessment and data processing for pavement design

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Summary

This paper is a succinct state of the art of the various methodologies used in the current road practice for traffic assessment and data processing for the design of road pavements. After a short introduction, presenting the general principles of traffic assessment methodology, the following procedures are considered: the actual methodology used in this county and the Highway Agency U.K. Finally a comparative analysis of these methodologies with the aim of establishing some useful correlations for the structural design of the pavements is presented.

KEYWORDS: traffic census, design traffic, forecast traffic, standard axles load, equivalence coefficients, pavement design.

1. INTRODUCTION

In this stage of development of structural design of road pavements at the level of world practice, it is necessary to establish some traffic correlations [1] between various methodologies of traffic assessment used in various countries. In this paper in relations with Figure 1 the following procedures for pavement design traffic assessment are analyzed in detail: Romanian methodology and the Highway Agency U.K.

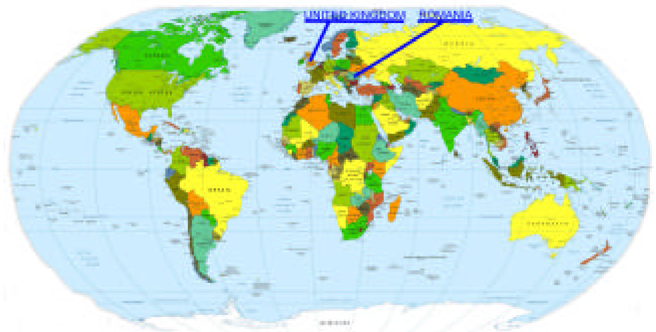


Figure 1. Location of studied country on the world map

By considering the specific vehicle fleet as described in Figure 2, we may observe that in comparison with the Romanian procedure, which has three classes of combined vehicles, the British are taking into consideration each vehicle class, represented by a single vehicle type.

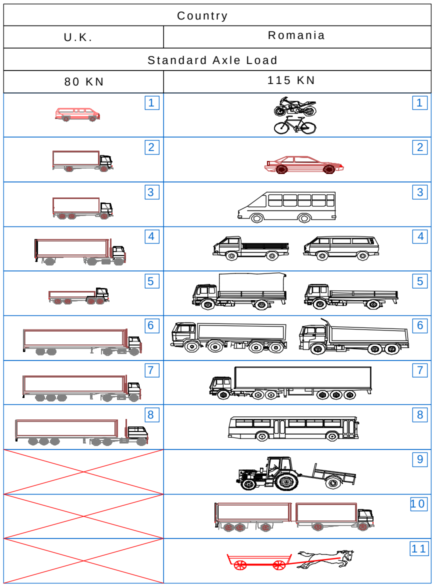


Figure 2. General table of vehicle census for the involved countries

2. ROMANIAN METHODOLOGY FOR TRAFFIC ASSESSMENT & DATA PROCESSING

In accordance with the Romanian norms [2] for the design of flexible and composite pavements the design traffic corresponding to the forecast period is expressed in standard axels loads of 115 kN, having the following characteristics:

- the load on double wheels: 57.5 kN
- pressure of contact: 0.625 MPa
- equivalent radius of impressioning area: 0.171 m

2.1. Design life period:

The design life period it is established within the first design stage, considering both the present traffic and its forecast evolution. Fifteen or twenty-year period is usually taken into consideration for the design life of a road, the Romanian norm specifying for new roads 15 - 20 year and for rehabilitations 10 – 15 year.

2.2. Traffic structure and intensity:

The traffic structure and intensity corresponding to a traffic census station are applied on the road section assigned to that station, according to road network division made at the latest general traffic census. At the important road works, as construction of new roads, express roads and motorways, planning of intersection, that impose knowledge of traffic streams on the whole roads networks, the traffic census data should be completed, by requirement, through traffic inquiries performed and processed within a specific traffic study. This is especially recommended in case of streets and county roads, rural roads and by-roads where no traffic census station worked or traffic re-distributions are anticipated.

2.3. Forecast traffic:

The minimal coefficients of evolution, based on the latest general traffic census for 2005 - 2025, on groups of vehicles, are given in Table 1. Values of these coefficients of evolution should be updated after every general traffic census undertaken by National Administration of the Roads. The traffic studies for the European roads under modernization or on others roads of heavy traffic, should consider within traffic structure, the vehicles with load axle ranged between 100 kN and 115 kN as result of growth of maximum limits of tonnage on single axle form 10.0 t to 11.0 t.

2.4. Equivalence coefficients for standard axles of different types of vehicles

The evolution coefficients of road traffic, on various vehicle groups, for period 2005 -2025, for a National Romanian road, according [3] are given in Table 1.

Table 1. The coefficients of traffic evolution for national roads [3]

Year	Vehicle group					
	2 axles trucks	3 - 4 axles trucks	Articulated trucks	Buses	Tractors & Special vehicles	Trucks & Trailers
2005	1,00	1,00	1,00	1,00	1,00	1,00
2010	1,33	1,32	1,22	1,21	1,30	1,31
2015	1,71	1,55	1,46	1,40	1,61	1,66
2020	2,20	1,77	1,71	1,60	1,94	1,98
2025	2,58	2,00	1,98	1,79	2,09	2,16

The coefficient for conversion of various vehicles in standard axle load of 115 kN is given in Table 2.

Table 2. The coefficients for conversion of the various vehicles in standard axles of 115 kN [3]

Type of pavement	Vehicle group					
	Trucks and derivatives with 2 axles	Trucks and derivatives with 3 - 4 axles	Articulated vehicles	Buses	Tractors & Special vehicles	Trucks & Trailers
Flexible	0,4	0,6	0,8	0,6	0,3	0,8

2.5. Design traffic:

The design traffic is expressed in millions standard axles of 115 kN (m.s.a.) and is established on the base of the annual average daily traffic, with relation 1:

$$N_c = 365 \times 10^{-6} \times p_p \times c \times \sum_{k=1}^{5} n_{ki} \times \frac{p_{kR} + p_{kF}}{2} \times f_{ek} \quad (1)$$

Where:

- Nc - the design traffic;
- 365 - number of calendar days in a year;
- p_p - the prospect period, in years;
- c_{rt} - coefficient of transversal repartition on traffic lanes, namely:
  - for roads with two and three traffic lanes = 0.50
  - for roads with four or more traffic lanes = 0.45
- n_{ki} - intensity of annual daily mean of vehicles from group k, according to traffic census results;
- p_{kR} - coefficient of evolution of vehicles from group k, corresponding to opening year of the road, yearR, obtained by interpolation;
- p_{kF} - coefficient of evolution of vehicles from group k, corresponding to the end of the prospect period (year F), obtained by interpolation;
- f_{ek} - equalization coefficient of vehicles from group k in standard axles of 115 kN, according to Table 1;

In case the data concerning the present and forecast AADT in standard axles of 115 kN are not available, the design traffic can estimated with the relation 2:

$$N_c = 365 \times 10^{-6} \times p_c \times c_{rt} \times \frac{n_{s.a.115R} + n_{s.a.115F}}{2} (m.s.a.) \quad (2)$$

Where:

- Nc - the design traffic;
- 365 - number of calendar days in a year;
- p_p - the prospect period, in years;
- c_{rt} - coefficient of transversal repartition on traffic lanes, namely:
  - for roads with two and three traffic lanes = 0.50
  - for roads with four or more traffic lanes = 0.45

- $n_{s.a.115R}$ - number of standard axles of 115 kN, corresponding to opening year of the road (year R) obtained by interpolation;
- $n_{s.a.115F}$ - number of standard axles of 115 kN, corresponding to the end of the prospect period (year F), obtained by interpolation.

2.6. Vehicle categories considered in the frame of traffic census

The following categories of vehicles described in accordance with Figure 3 are considered when performing the traffic census in this country:

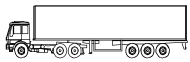
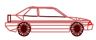
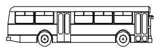
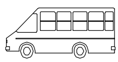
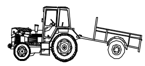
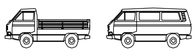
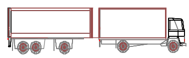
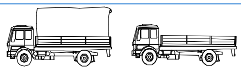
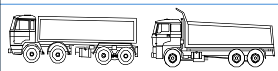
Vehicle Category	Standard Axle Load - 115 KN	Vehicle Category	Standard Axle Load - 115 KN
Bicycles / Motorcycles	 1	Articulated vehicles (TIR) & tugs with trailer with more than 4 axes	 7
Passenger car	 2	Buses & Coaches	 8
Minibus with max. 8 +1 seats	 3	Tractors with / without trailers & special vehicles	 9
Trucks & Commercial vehicle with MTMA <= 3.5 t	 4	Trucks with trailers with 2, 3 or 4 axle (road train)	 10
Trucks & derivatives with 2 axles	 5	Vehicle with animal traction	 11
Trucks & derivatives with 3 & 4 axles	 6		

Figure 3. Vehicle category for traffic census in Romania

In the Annex 1 a suggestive example of application of this methodology, based on the real traffic data recorded on a Romanian National Road, is given.

3. U.K. METHODOLOGY FOR TRAFFIC ASSESSMENT & DATA PROCESSING

Road loading takes many different forms, from a bicycle to multi-axle truck and trailer combinations. Traffic Analysis [4], [5] can be divided into two well defined areas: Traffic Volume and Traffic Loading. The pavement engineers are interested mainly in determining the loading on the road to be carried forward to the Pavement Design.

3.1. Traffic Volume

The traffic volume values are used to determine the road width only. According with literature [4], [5] in “relation to the volume of traffic using the road, the passenger car is adopted as the standard unit and other vehicles are assessed in terms of passenger car units - pcu”. The classification of vehicles in pcu's is described in the next table:

Table 3. The classification of vehicles in pcu's [4], [5]

Type of Vehicle	Rural	Urban
Private cars, motor cycle combinations, taxis and light private goods vehicles up to 1.5t unlade	1	1
Motorcycles (solo), scooters and mopeds	1	0.75
Goods vehicles over 1.5t unlade weight	3	2

3.2. Traffic Loading

The traffic loading taken into consideration for pavement design is expressed in Millions of Standard Axles. The role of the designer is to provide a proper thickness of pavement to carry the expected loads without deterioration of the network during the design life. It is recognized that damage increases with axle loading and for this reason only the effect of vehicles over 1.5t are considered. According British practice these heavy vehicles are divided into the following categories as shown in Figure 4:

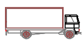
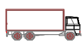
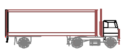
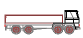
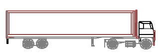
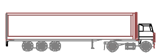
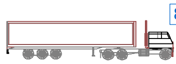
Vehicle Classes	Vehicle Category	Standard Axle Load 80 KN
Buses and Coaches	PSV	 1
2 - axle rigid	OGV 1	 2
3 - axle rigid		 3
3 - axle articulated	OGV 2	 4
4 - axle rigid		 5
4 - axle articulated		 6
5 - axle articulated		 7
6 (or more) axle articulated		 8

Figure 4. Classes and categories of vehicles for traffic census in U.K.

3.3. Assessment Methods

There are two methods of assessing the road loading on the pavement: standard and non-standard. The standard method is used when the design is for a new road.

3.3.1. Standard Method

The standard method it is used for all new road design. It involves the determination of the total flow of commercial vehicles per day - cv/day and the OGV2 percentage.

The following procedures can be used:

- If the road is new then the estimation is performed by traffic engineer.
- For maintenance design a classified count is carried out over either a 12, 16 or 24 hour period

These figures must then be processed into a figure representing the loading on the road over the design life in terms of millions of standard axles – msa, as shown in Figure 5 & Figure 6, which are used in the pavement design directly to calculate the pavement thickness. The design charts used depend on the type of pavement, the number of carriageways and the chosen design life.

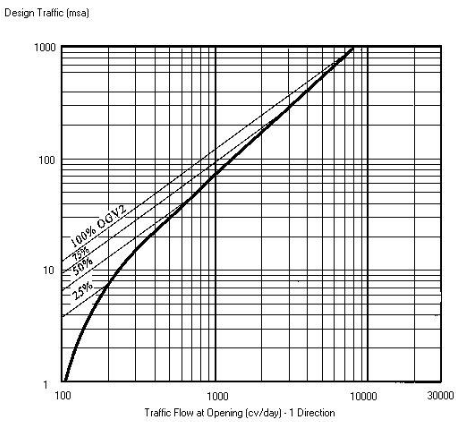


Figure 5. Design Traffic for Rigid, Rigid Composite and Flexible Pavements (40 year life) - Single Carriageway [4], [5]

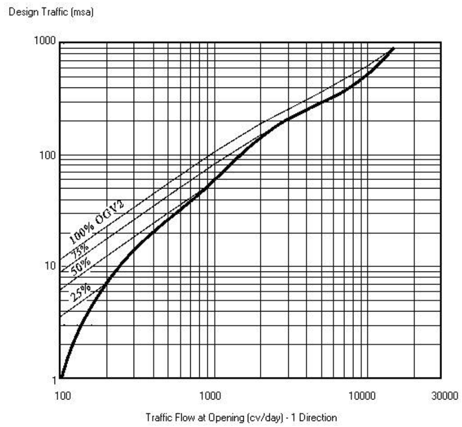


Figure 6. Design Traffic for Rigid, Rigid Composite and Flexible Pavements (40 year life) - Dual Carriageway [4], [5]

3.3.2. Non-Standard Method

The non standard method is used in structural assessment and maintenance design. The initial loadings for each category expressed in terms of an average annual daily flow - AADF, is necessary to determine the growth factor - G, the wear factor - W and the design life - Y years. The growth factor is obtain from design charts shown in Figure 7&8, function of design period and growth rate and whether the road is new or not.

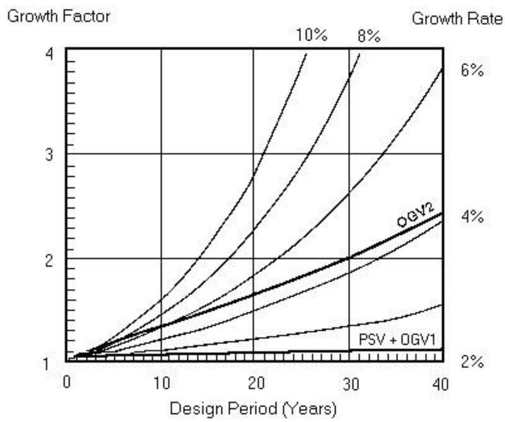


Figure 7. Derivation of Growth Factor for New Roads [4], [5]

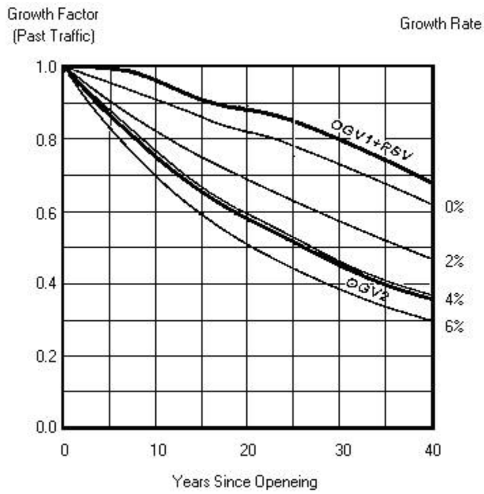


Figure 8. Derivation of Growth Factor for Past Traffic [4], [5]

The bold lines on the charts represent the National Road Traffic Forecast - NRTF values and may be used when no other data are available. Based on these factors the loading in *m.s.a.* can be determined by using the relation:

$$\text{Traffic (m.s.a.)} = 365*(AADF)*Y*G*W*10^{-6} \quad (3)$$

In the Annex 2 a suggestive example of application of the UK Highway Agency methodology, based on the same traffic considered in the example from Annex 1 is given.

5. CONCLUSIONS

- The results obtained using these two methodologies in the applications described in Annex 1 & Annex 2 are given in Table 4.

Table 4. Synthesis of the design traffic obtained with the investigated methods

	Romanian methodology Nc (m.s.a. 115 kN)	UK/Highway Agencies Traffic (m.s.a. 80 kN)
Design Traffic - Expressed in specific axle loads	5,50	16,93
Design Traffic - Expressed in 80 kN axle loads	**22,00**	**16,93**

- In order to compare these results it was necessary to convert the design traffic expressed in m.s.a. of 115 kN in equivalent number of standard axle of 80 kN, by using the ASSHTO conversing coefficient according literature [6].
- Comparing this traffic design we conclude that the difference of 5 m.s.a. may be caused by the specific of the methodologies used for this assessment, the British methodology is more precise and takes into considerations every type of vehicle from the entire traffic fleet.
- In order to better clarify the reason of appearance of such differences, this study is intending to develop new applications in the near future, including in the study other methods such as ME-PDG [7] and APA [8].

Annexes:

Annex 1: Example of applications of Romanian methodology for assessment of the design traffic;
Annex 2: Example of applications of U.K. methodology for assessment of the design traffic;

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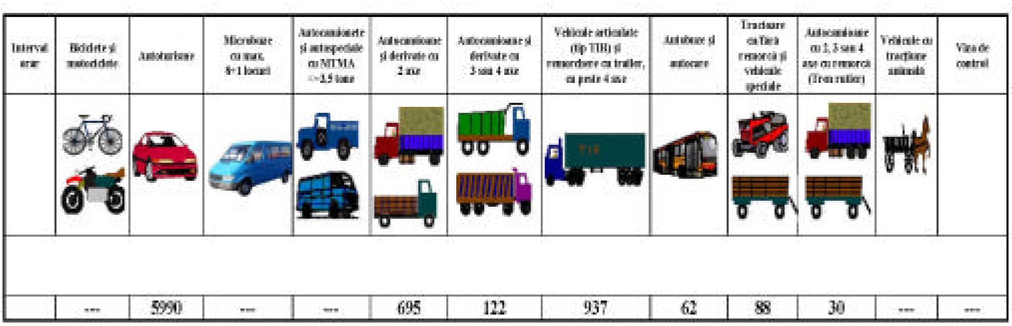
Annex 1: Example of applications of Romanian methodology for assessment of the design traffic

During the traffic census, made on the National road, the following categories of vehicles have been recorded:



Interval orar	Biciclete şi motociclete	Autoturisme	Microbuze cu max. 8+1 locuri	Autocamionete şi autospeciale cu MTMA <=3,5 tone	Autocamioane şi derivate cu 2 axe	Autocamioane şi derivate cu 3 sau 4 axe	Vehicule articulate (tip TIR) şi remorchere cu trailer, cu peste 4 axe	Autobuze şi autocare	Tractoare cu/fără remorcă şi vehicule speciale	Autocamioane cu 2, 3 sau 4 axe cu remorcă (Tren rutier)	Vehicule cu tracţiune animală	Viteza de control
	---	5990	---	---	695	122	937	62	88	30	---	---

According [2], the assessment of traffic is shown in Table 1.

Table 1. Assessment of the design traffic

Grupa de vehicule	$n_{k\,05}$	$p_{k\,10}$	$p_{k\,25}$	$(p_{k\,10} + p_{k\,25}) \times 0,5$	f_{ek}	col.1 x col.4.x col.5 o.s.115
Trucks and derivatives with 2 axles	695	1,33	2,58	1,955	0,40	544
Trucks and derivatives with 3-4 axles	122	1,32	2,00	1,66	0,60	122
Articulated Vehicles	937	1,22	1,98	1,60	0,80	1200
Buses	62	1,21	1,79	1,50	0,60	56
Tractors & Special vehicles	88	1,30	2,09	1,695	0,30	45
Trucks & Trailers	30	1,31	2,16	1,735	0,80	42
Total axels standard 115 kN						2009

$N_c = 365 \times 10^{-6} \times 15 \times 0,5 \times 2009 = 5,50$ m.o.s.

Annex 2: Example of applications of U.K. methodology for assessment of the design traffic

A traffic survey for the same national road taken in Annex 1 gives the following count data:

- Buses & Coaches 62
- 2 Axle rigid 695
- 3 Axle rigid 50
- 3 Axle articulated 537
- 4 Axle rigid 50
- 4 Axle articulated 400
- 5 Axle or more 22

Using the structural assessment & maintenance method we calculate the design traffic expressed in m.s.a. for a design life of 15 years given the following:

Category	Growth Factor	Wear Factor
OGV1 & PSV	1.05	0.6
OGV2	1.3	3.0

This is a simply case of inserting the data into a standard template and doing the math’s to produce the figures below. The traffic in msa comes from the formula with the values as shown and a design life Y=15 years:

$$\text{Traffic (msa)} = 365*(\text{AADF})*Y*G*W*10^{-6}$$

Vehicle Type	AADF		Growth Factor (G)	Wear Factor (W)	Traffic (msa)
Buses & Coaches	62	PSV & OGV1	1.20	0.6	5,30
2 Axle rigid	695	= 1344			
3 Axle rigid	50				
3 Axle articulated	537				
4 Axle rigid	50	OGV2	1.50	3.0	11,63
4 Axle articulated	400	=472			
5 Axles or more	22				
			Design Traffic = 16,93 msa		

This value of vehicle loading 16.93 is expressed in standard axle of 80 KN according U.K. norm and then is carried forward to the pavement design.

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Stability in bending and axial compression of steel members in accordance with EN 1993

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Summary

This paper presents the stability verification methodology of uniform members with double symmetric cross section subjected to bending and axial compression in accordance with Eurocode 3: Design of steel structures (EN 1993-1-1:2003: General rules and rules for buildings; EN 1993-1-5: 2004: Plated structural elements).

The numerical example also detailed in the paper, concerning the buckling verification of a member with a double T symmetric cross section, subjected to monoaxial bending and axial compression is useful to understand the design methodology.

KEYWORDS: Eurocode 3, stability, bending, axial compression, double symmetric cross section.

1. INTRODUCTION

The general stability of a steel member subjected to bending and compression is a complex phenomenon where the flexural and torsional buckling, the lateral buckling of the compression flange and their interaction have been taken into account.

The cross sections are divided into four cross section classes with the role of identifying the extend to which the resistance and rotation capacity of the cross section is limited by its local buckling resistance.

In Class 4 cross sections effective width may be used to make the necessary allowances for reduction in resistance due to the effects of local buckling. For a Class 4 cross section it should also be determined the possible shift e_N of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross section and the resulting additional moment:

$$\Delta M_{Ed} = N_{Ed} \cdot e_N$$

2. UNIFORM MEMBERS IN BENDING AND AXIAL COMPRESSION

In accordance with [1], unless second order analysis is carried out, the stability of uniform members with double symmetric cross sections which are subjected to combined bending and compression should satisfy:

$$\frac{N_{Ed}}{\chi_y \dfrac{N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y.Ed} + \Delta M_{y.Rd}}{\chi_{LT}\dfrac{M_{y.Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z.Ed} + \Delta M_{z.Ed}}{\dfrac{M_{z.Rk}}{\gamma_{M1}}} \le 1 \qquad (1.a)$$

$$\frac{N_{Ed}}{\chi_z \dfrac{N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y.Ed} + \Delta M_{y.Rd}}{\chi_{LT}\dfrac{M_{y.Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z.Ed} + \Delta M_{z.Ed}}{\dfrac{M_{z.Rk}}{\gamma_{M1}}} \le 1 \qquad (1.b)$$

where:

$N_{Ed}, M_{y.Ed}, M_{z.Ed}$ – the design values of the compression force and the maximum moments about the y-y and z-z axis;

$\Delta M_{y.Ed}, \Delta M_{z.Ed}$ – the moments due to the shift of the centroidal axis for class 4 sections;

χ_y, χ_z – the reduction factors due to flexural buckling; χ_{LT} – the reduction factor due to lateral torsional buckling ;

$k_{yy}, k_{yz}, k_{zy}, k_{zz}$ – the interaction factors which may be obtained from Annex A (alternative method 1) or from Annex B 9 (alternative method 2).

The characteristic resistances $N_{Rk} = A_i \cdot f_y$; $M_{Rk} = W \cdot f_y$ and $\Delta M_{Ed} = e_N \cdot N_{Ed}$ will be evaluated in accordance with Table 1.

Table 1

CLASA	1	2	3	4
A_i	A	A	A	A_{eff}
W_y	$W_{pl.y}$	$W_{pl.y}$	$W_{el.y}$	$W_{eff.y}$
W_z	$W_{pl.z}$	$W_{pl.z}$	$W_{el.z}$	$W_{eff.z}$
$\Delta M_{y.Ed}$	0	0	0	$e_{N.y} \cdot N_{Ed}$
$\Delta M_{z.Ed}$	0	0	0	$e_{N.z} \cdot N_{Ed}$

The reduction factor χ_{TF} is determined function of the torsional –flexural non-dimensional slenderness, $\bar{\lambda}_{TF}$:

$$\bar{\lambda}_{TF} = \sqrt{\frac{A_i f_y}{N_{cr}}} \text{ ; where: } A_i = \begin{cases} A & - \text{sections Class 1; 2; 3} \\ A_{eff} & - \text{section Class 4} \end{cases} \tag{2}$$

$$N_{cr} = \min.(N_{cr.T}; N_{cr.TF}) \tag{3}$$

3. WORKING EXAMPLE

The general stability in accordance with EN 1993-1-1:2003 of a uniform member with a double symmetric cross section subjected to bending and axial compression is checked. The following design data are given:

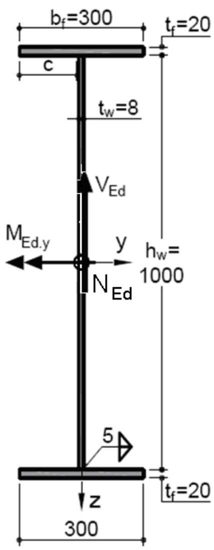


Figure 1. Member cross section

Steel S355 ($f_y = 355 \text{ N/mm}^2; \varepsilon = 0.81$)

Loadings:

- $N_{Ed} = 1200$ kN
- $M_{Ed.y} = 500 \text{ kN} \cdot \text{m}$
- $V_{Ed} = 400$ kN

Lengths:

L=12.0 m

$l_{fy} == L_{cr.y} = 12.0$ m

$l_{fz} == L_{cr.z} = L_{cr.LT} = 4.0$ m

3.1. Cross section characteristics

$A = 200 \text{ cm}^2$; $I_y = 378\,817 \text{ cm}^4$; $W_{el.y} = 7\,285 \text{ cm}^3$; $W_{pl.y} = 8\,120 \text{ cm}^3$;

$I_z = 9\,004 \text{ cm}^4$; $W_{el.z} = 600.3 \text{ cm}^3$; $W_{pl.z} = 916 \text{ cm}^3$, $I_\omega(I_w) = 2.341 \cdot 10^7 \text{ cm}^6$

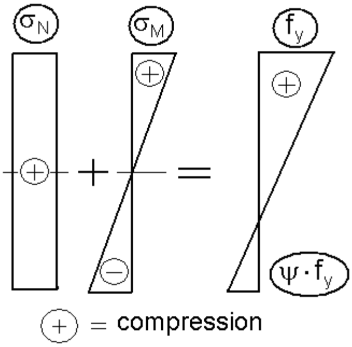
Cross section Class

The flange is an outstand element under uniform compression ($\psi = 1$; $k_\sigma = 0.43$).

$$\frac{c}{t_f} = \frac{\left[b_f - \left(t_w + 2\sqrt{2} \cdot a_w\right)\right]/2}{t_f} = \frac{\left[300 - \left(8 + 2\sqrt{2} \cdot 5\right)\right]/2}{20} = 6.95 < 9 \cdot \varepsilon = 7.29 \Rightarrow$$

The flange is Class 1.

The web is subjected to bending and compression. The stress ratio at ULS is given by (Figure 2):



$$\psi = \frac{2\,N_{Ed}}{Af_y} - 1 = \frac{2 \cdot 1200 \cdot 10^2}{200 \cdot 3550} - 1 = -0.66 > -1$$

$$\frac{c}{t_w} = 123.2 > \frac{42 \cdot \varepsilon}{0.67 + 0.33\psi} = 75.2 \Rightarrow$$

The web is Class 4

Figure 2. The stress ratio

The section is Class 4 $\Rightarrow$ The verification of the member will be based on the elastic resistance of the effective cross-section.

Effective cross section

Effective area: the effective area of the cross section is determined under compression only - EN 1993-1-1. § 6.2.9.3(2).

The flange is Class 1 $\Rightarrow A_{f.eff} = A_f$

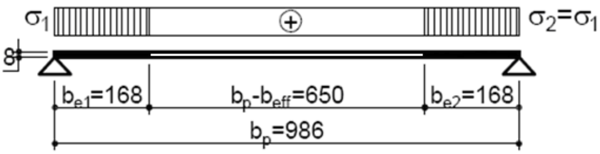
The web is an internal element subjected to uniform compression, $\psi = 1$; $k\sigma = 4$, Figure 3.

$$\frac{c}{t_w} = \frac{h_w - 2\sqrt{2} \cdot a_w}{t_w} = \frac{1000 - 2\sqrt{2} \cdot 5}{8} = 123.2 > 42\varepsilon = 34 \Rightarrow \text{Class 4}$$

Slenderness: $$\bar{\lambda}_p = \frac{c/t_w}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} = \frac{986/8}{28.4 \cdot 0.81 \cdot \sqrt{4}} = 2.68 > 0.673$$

Reduction factor: $\rho = \dfrac{\overline{\lambda}_p - 0.055(3+\psi)}{\overline{\lambda}_p^2} = \dfrac{2.68 - 0.055(3+1)}{2.68^2} = 0.34 < 1$

Effective width: $b_{eff} = \rho \cdot c = 0.34 \cdot 986 = 336\ \text{mm}$; $b_{e1} = b_{e2} = 0.5 \cdot b_{eff} = 168\ \text{mm}$



$\oplus$ – compression

Figure 3

Figure 4 presents the effective section taking into account the compression only.

It is obtained: $A_{eff} = 200 - 0.8 \cdot 65 = 148\ \text{cm}^2$

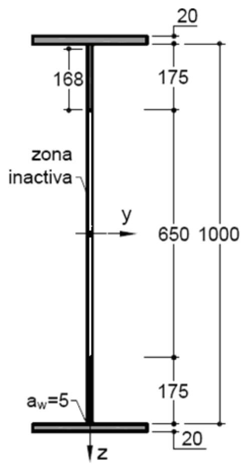


Figure 4. Effective section taking into account the compression only

The effective elastic modulus is determined under bending only - EN 1993-1-1. § 6.2.9.3(2).

The flange is Class 1 $\Rightarrow A_{f.eff} = A_f$

The web is an internal element subjected to bending, $\psi = -1$; $k\sigma = 23.9$, Figure 5.

$$\frac{c}{t_w} = 123.2 > 124\varepsilon = 100.44 \Rightarrow \text{Class 4}$$

Slenderness: $\bar{\lambda}_p = \frac{c/t_w}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} = \frac{986/8}{28.4 \cdot 0.81 \cdot \sqrt{23.9}} = 1.1 > 0.673$

Reduction factor: $\rho = \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\bar{\lambda}_p^2} = \frac{1.1 - 0.055(3-1)}{1.1^2} = 0.82 < 1$

Effective width: $b_{eff} = 404\ \text{mm}$; $b_{e1} = 0.4 \cdot b_{eff} = 162\ \text{mm}$; $b_{e2} = 0.6 \cdot b_{eff} = 242\ \text{mm}$

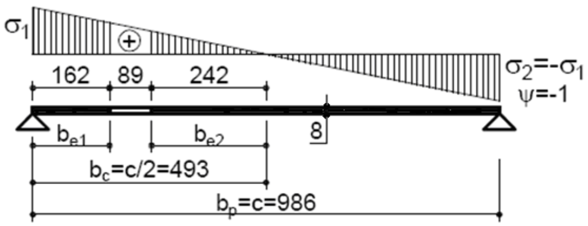


Figure 5

Figure 6 presents the effective section taking into account the bending only. It is obtained: $I_{eff.y} = 3.66 \cdot 10^5\ \text{cm}^4$; $W_{eff.y} = 6\,973\ \text{cm}^3$.

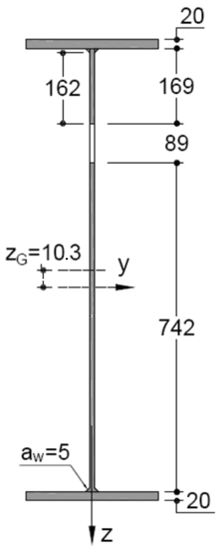


Figure 6. Effective section taking into account the bending only

3.2. Verification of the buckling resistance

In this case we have:

$$e_{Ny} = e_{Nz} = 0 \Rightarrow \Delta M_{y.Rd} = \Delta M_{z.Rd} = 0\,;\; M_{z.Ed} = 0$$

The relations (1.a,b) become:

$$\frac{N_{Ed}}{\chi_y \dfrac{N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y.Ed}}{\chi_{LT}\dfrac{M_{y.Rk}}{\gamma_{M1}}} \le 1 \quad \text{and} \quad \frac{N_{Ed}}{\chi_z \dfrac{N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y.Ed}}{\chi_{LT}\dfrac{M_{y.Rk}}{\gamma_{M1}}} \le 1$$

The k_{yy} and k_{zy} factors will be calculated using the Annex A of EN 1993-1-1.

Buckling about y-y - axis

$$N_{cr.y} = \pi^2\frac{EI_y}{L^2_{cr.y}} = \pi^2\frac{2.1\cdot 10^6\cdot 3.788\cdot 10^5}{1200^2}\cdot 10^{-2} = 5.45\cdot 10^4 \text{ kN}$$

$$\bar{\lambda}_y = \sqrt{\frac{A_{eff}\,f_y}{N_{cr.y}}} = \sqrt{\frac{148\cdot 3550}{5.45\cdot 10^6}} = 0.31 \quad \Rightarrow \chi_y = 0.96 \quad (\text{curve „b”.} \;\; t_f < 40)$$

It is obtained:

$$N_{by.Rd} = \chi_y\frac{N_{Rk}}{\gamma_{M1}} = \chi_y\frac{A_{eff}\,f_y}{\gamma_{M1}} = 0.96\frac{148\cdot 3550}{1.1}\cdot 10^{-2} = 4\,585 \text{ kN}$$

Buckling about z-z - axis

$$N_{cr.z} = \pi^2\frac{EI_z}{L^2_{cr.z}} = \pi^2\frac{2.1\cdot 10^6\cdot 9\,004}{400^2}\cdot 10^{-2} = 1.166\cdot 10^4 \text{ kN}$$

$$\bar{\lambda}_z = \sqrt{\frac{A_{eff}\,f_y}{N_{cr.z}}} = \sqrt{\frac{148\cdot 3550}{1.166\cdot 10^6}} = 0.67 \quad \Rightarrow \chi_z = 0.74 \quad (\text{curve „c”})$$

It is obtained:

$$N_{bz.Rd} = \chi_z\frac{N_{Rk}}{\gamma_{M1}} = \chi_z\frac{A_{eff}\,f_y}{\gamma_{M1}} = 0.74\frac{148\cdot 3550}{1.1}\cdot 10^{-2} = 3\,535 \text{ kN}$$

Lateral torsional buckling

The critical moment for a doubly symmetrical section ($L_{cr.LT} = 4.0$ m):

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L_{cr.LT}^2} \sqrt{\frac{I_w}{I_z} + \frac{L_{cr.LT}^2 GI_t}{\pi^2 EI_z}} =$$

$$= 1 \cdot \frac{\pi^2 2.1 \cdot 10^6 \cdot 9004}{400^2} \sqrt{\frac{2.341 \cdot 10^7}{9004} + \frac{400^2 \cdot 0.807 \cdot 10^6 \cdot 177}{\pi^2 2.1 \cdot 10^6 \cdot 9004}} \cdot 10^{-4} = 6079 \text{ kNm}$$

where: $C_1 = 1$ - for M constant $(\psi = +1)$

The slenderness for lateral torsional buckling:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{eff.y} f_y}{M_{cr}}} = \sqrt{\frac{6973 \cdot 3350}{6079 \cdot 10^4}} = 0.62 \quad \Rightarrow \chi_{LT} = 0.69 \text{ (curve „d" - } \alpha_{LT} = 0.76\text{)}$$

It is obtained:

$$M_{b.Rd} = \chi_{LT} \frac{M_{y.Rk}}{\gamma_{M1}} = \chi_{LT} \frac{W_{eff.y} f_y}{\gamma_{M1}} = 0.69 \frac{6973 \cdot 3550}{1.1} \cdot 10^{-4} = 1553 \text{ kNm}$$

Factors μ_{yy} and μ_{zy} (Annex A – EN 1993-1-1)

$$k_{yy} = C_{my} C_{m.LT} \frac{\mu_y}{1 - \frac{N_{Ed}}{N_{cr.y}}} = 1.022 \cdot C_{my} C_{m.LT};$$

$$k_{zy} = C_{my} C_{m.LT} \frac{\mu_z}{1 - \frac{N_{Ed}}{N_{cr.y}}} = 1 \cdot C_{my} C_{m.LT}$$

where:

$$\mu_y = \frac{1 - \frac{N_{Ed}}{N_{cr.y}}}{1 - \chi_y \frac{N_{Ed}}{N_{cr.y}}} = \frac{1 - \frac{1200}{5.45 \cdot 10^4}}{1 - 0.97 \frac{1200}{5.45 \cdot 10^4}} = 1$$

$$\mu_z = \frac{1 - \frac{N_{Ed}}{N_{cr.z}}}{1 - \chi_z \frac{N_{Ed}}{N_{cr.z}}} = \frac{1 - \frac{1200}{1.166 \cdot 10^4}}{1 - 0.80 \frac{1200}{1.166 \cdot 10^4}} = 0.98$$

It is checked the relation which defines the susceptibility to torsional deformations:

$$\bar{\lambda}_0 \le \bar{\lambda}_{0.\lim} = 0.2\sqrt{C_1}\,\sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr.z}}\right)\left(1-\frac{N_{Ed}}{N_{cr.TF}}\right)} =$$

$$= 0.2\sqrt{1}\,\sqrt[4]{\left(1-\frac{1200}{1.166\cdot 10^4}\right)\left(1-\frac{1200}{1.64\cdot 10^4}\right)} = 0.19$$

where: $\bar{\lambda}_0 = \lambda_{LT} = 0.62$ - the slenderness for lateral torsional buckling for an uniform moment.

For a doubly symmetrical section $N_{cr.TF} = N_{cr.T}$:

$$N_{cr.TF} = N_{cr.T} = \frac{A}{I_0}\left(GI_t + \frac{\pi^2 EI_w}{L_{LT}^2}\right) =$$

$$= \frac{200}{3.87\cdot 10^5}\left(0.807\cdot 10^6\cdot 177 + \frac{\pi^2\, 2.1\cdot 10^6\cdot 2.341\cdot 10^7}{400^2}\right) = 1.64\cdot 10^6 \text{ daN}$$

$$I_0 = I_y + I_z = 3.878\cdot 10^5 \text{ cm}^4\,;\quad I_t = \frac{1}{3}\left(2\cdot 30\cdot 2^3 + 100\cdot 0.8^3\right) = 177 \text{ cm}^4$$

For $\bar{\lambda}_0 = 0.62 > \bar{\lambda}_{0.\lim} = 0.19$, $C_{my} = C_{my.0} + \left(1 - C_{my.0}\right)\frac{\sqrt{\varepsilon_y}\, a_{LT}}{1+\sqrt{\varepsilon_y}\, a_{LT}} = 1$

where:

$$\varepsilon_y = \frac{M_{y.Ed}}{N_{Ed}}\frac{A_{eff}}{W_{eff.y}} = \frac{500\cdot 10^4}{1200\cdot 10^2}\frac{148}{7031} = 0.88$$

$$a_{LT} = 1 - \frac{I_t}{I_y} \approx 1$$

For $\psi_y = 1$ (uniform moment), the equivalent uniform moment factor is:

$$C_{my.0} = 0.79 + 0.21\cdot\psi_y + 0.36\left(\psi_y - 0.33\right)\frac{N_{Ed}}{N_{cr.y}} = 1$$

Factor $C_{m.LT}$:

$$C_{m.LT} = C_{my}^2\,\frac{a_{LT}}{\sqrt{\left(1-\frac{N_{Ed}}{N_{cr.z}}\right)\left(1-\frac{N_{Ed}}{N_{cr.T}}\right)}} = 1.09 > 1$$

It is obtained: $k_{yy} = 1.11$; $k_{zy} = 1.09$

The interaction formulae for the member buckling which is subjected to bending and compression are:

$$\frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y.Ed}}{\chi_{LT} \frac{M_{y.Rk}}{\gamma_{M1}}} = \frac{1200}{4585} + 1.11 \frac{500}{1553} = 0.62 < 1$$

$$\frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y.Ed}}{\chi_{LT} \frac{M_{y.Rk}}{\gamma_{M1}}} = \frac{1200}{3535} + 1.09 \frac{500}{1553} = 0.69 < 1$$

4. FINAL REMARKS

The general stability of the steel member subjected to bending and axial compression in accordance with EN 1993-1-1 takes into account the following aspects:

- the flexural buckling by using the reduction factors χ_y and χ_z;

- the lateral torsional buckling by using the reduction factor χ_{LT};

- the effective properties of the cross section;

- the moments due to the shift of the centroidal axis for class 4 sections;

- the interaction between different loadings by using the interaction factors $k_{yy}, k_{yz}, k_{zy}, k_{zz}$.

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The robustness of engineering structures and ways to develop this concept in analyzing and designing road structures

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Summary

The paper is a state-of-the-art about the concept of the robustness of engineering structures. This concept is a relatively new theme in structural engineering. The safety of a structure is one of the main issues of design and it is shown that current design methods are inadequate to prevent progressive collapse (Starossek and Wolff 2005).

Robustness represents the property of structures to resist unusual or unforeseen actions without suffering from excessive defect or loss of function.

It can also be described as the ability of structures to resist progressive collapse.

In the civil engineering field robustness of structures had been studied for the first time back in 1968 due to an accident in eastern London – partial collapse of the 22-floor Ronan Point building. Later, in the 1980’s and 1990’s the interest on this issue decreased, but with the bombing, terrorist attacking and collapse of the World Trade Center Towers in New York the problem of structure robustness came into attention once again(COST ACTION 2006).

In the road field, structural robustness has not been properly approached yet, the study of this concept being imperatively necessary in order to avoid different types of road structures collapse.

KEYWORDS: collapse, road, robustness, Ronan Point, structure.

1. INTRODUCTION

Due to the fact that it requires an extreme loading to cause damage and also a structure that isn’t strong, solid enough, progressive collapse is a quite rare event. However, the recent, unforeseen events of terrorism that humanity has dealt with in the past few years stressed the urgent need for rational approaches on this matter in order to reduce to the minimum the risks on environment, assets and most of all on people.

A number of unusual accidents has brought the concept of structural robustness into scientists attention and also increased the need to go deep into this matter.

1.1 Ronan Point Building

In 1968 a gas explosion produced a partial collapse at the Ronan Point building from eastern London. The collapse started at the 18th floor where the opposite corner walls of the apartment were blown away by the gas explosion. As they were the only support for the walls directly above it created a chain reaction in which floors 19 to 22 fell on floor 18 while the resulted loading on floor 18 caused the demolishing of the rest of the floors down to the ground. The explosion from 1986 demonstrates that poor workmanship was present in every joint of the building.

Figure 1. Ronan Point building

1.2. Charles de Gaulle, France 2004

Another case referring to an industrial building is the tragic collapse of one of Paris’ Charles de Gaulle terminal.

On May 2004 a 33-foot long section of a terminal's ceiling collapsed, killing four people. The experts pointed out that there were a series of causes for the collapse, rather than a single fault. The investigation found out that the concrete arched roof was not resilient enough and also some openings in it weakened the structure.

Before the collapse, a crack appeared in the departure lounge roof at the point where an intermediate steel section connected the exterior glass shell to the inner concrete shell. Concrete began to fall and the southern lateral supporting beam collapsed which brought the entire arched-section down

One of the causes was that the steel sections were embedded too deeply into the concrete. "The report also cited inadequate or badly positioned reinforcing within the concrete. A lack of redundancy meant that stress was carried to the weakest points of the structure. The horizontal concrete beams on which the shell rested were weakened by the passage of ventilation ducts. Finally, the exterior metal structure was not sufficiently resistant to temperature changes. On the morning of the collapse the temperature dropped sharply to 4.1° C, from 25° C during the week." (Claire Downey 2005)

Figure 2. Terminal 2E at Paris' Charles de Gaulle airport

1.3. Minneapolis Bridge Collapse, Minnesota USA, 2007

On August 1, 2007, the central span of the bridge suddenly gave away, followed by the adjoining spans. The structure and deck collapsed into the river, the south part overturning 25 m eastward in the process.

Figure 3. Gusset Plate fractured

This image, shows a fracture in a gusset plate, fracture that played a major role in the collapse. (National Transportation Safety Board photo)

On January 2008, the investigation of the National Transportation Safety Board had determined that the bridge's design steel gusset plates were undersized and unsuited to support the intended load of the bridge, load that in time had increased. The under-sized gusset plates measured 13 mm thick. Contributing to that was the fact that 51 mm of concrete were added to the road surface over the years, increasing by 20% the dead load of the bridge and also the enormous weight of construction equipment and material resting on it just above its weakest point. The load was estimated at 262,000 kg consisting of sand, water, and vehicles. Also, the National Transportation Safety Board determined that corrosion was not a significant factor.

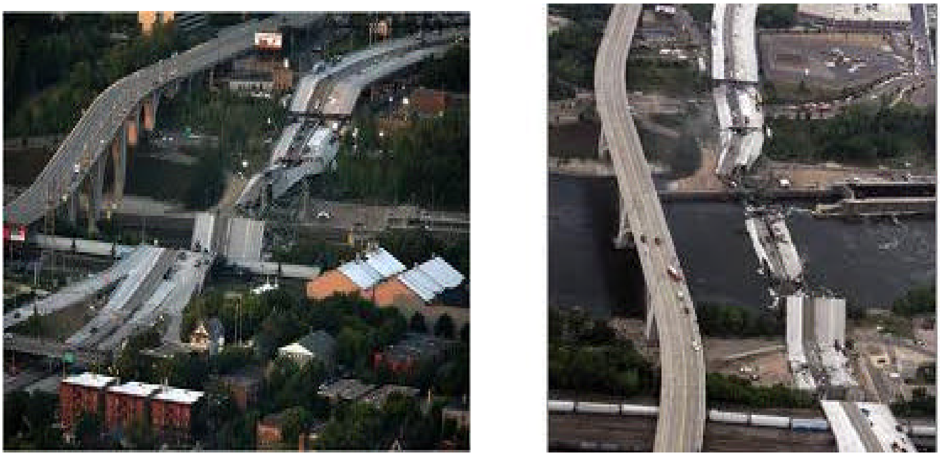

Figure 4. Minneapolis Bridge collapse

2. STRUCTURAL ROBUSTNESS IN ROAD FIELD

In the road field, robustness could be defined as the ability to deal with exceptional situations such as serious accidents, evacuations, or natural phenomena like earthquakes or floods.

“An adequate network first of all is aimed at minimizing the risk of network overloading. On the other hand, if overloading does occur, efforts should be directed at minimizing the consequences”(Immers et al. 2004)

There are two main issues that best characterize the concept of road network robustness. First of all, in order to reduce costs and human losses caused by disruptions of a road, it is important to maintain its main functions as much as possible after a break happened, and secondly, to repair the damage and recover the functions as soon as possible, in order to avoid complete collapse.

In relation with figure 5 there are two types of road structures:

2.1. Flexible Road Structures

This type of structure is composed of foundation layers made of different granular materials and asphaltic pavements.

2.2. Rigid Road Structures

This type of structure has foundation layers made of granular materials and concrete pavement.

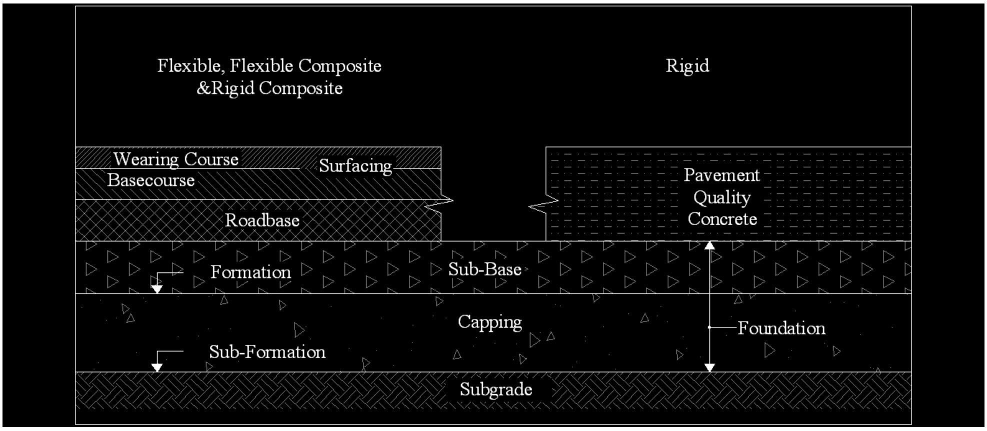

Figure 5. Typical Pavement

The robustness of this road structures "depends on several factors: redundancy, a network structure which affects the interdependency of a network and the quality of its service components, resilience and flexibility"(Immers et al. 2004).

3. CONCLUSIONS

Most of the serious accidents that happened in the last 40 years, accidents that generated progressive collapse, pointed out the importance and also the necessity to go deeply into the concept of structural robustness.

Although there is little information on robustness in expert writings, network robustness remains an important topic for road networks. The issues network robustness deal with underline most of all the capacity of a road network to function properly, at the same time being able to face unpredictable, exceptional incidents.

After a thorough analysis of the concept of robustness in different fields of science, the main aspects of a robustness evaluation in the road structures are as follows: a complete, clear system definition, identification of the feature we want the system to preserve, identification of specific risks and finally analysing the consequences of failure within the system.

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Traffic and transport infrastructure impact assessment on habitat

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Summary

This presentation highlight the problem of habitat fragmentation and of the negatives impacts of transportation infrastructure .
Mitigation , compensation as well as innovative projects undertaken with this purpose are presented.

KEYWORDS: fragmentation, habitat, mitigation, impacts.

1. INTRODUCTION

Transport activity and road infrastructure work on the environment and particularly on the habitat, affecting the ecology of related areas by its adverse effects [1].

This paper aims to present the current state of and various modalities of evaluation of such unwanted phenomena by, taking proper measures which may lead to the achievement of friendly environmentally transport and sustainability avoiding fragmentation and habitat destruction.

Finally the paper presents some proposals for investigation and research of these issues in different geographical areas, through the implementation of modern methodologies , assessment , mitigation and planning of these phenomena.

2. HABITAT FRAGMENTATION THROUGH TRANSPORT NETWORK.

The transport network divides the natural habitat into increasingly smaller areas, isolating them and creating barriers between them. This can lead to two primary effects on wildlife [1]:

- reduces and separates areas of habitat of vital populations of many important species;

- the isolation of remaining natural areas, which being separated from each other, does not allow wildlife to move from one area to another.

With no ability to move between habitat areas vulnerable species may reach local or regional extinctions. Through these processes the fragmentation of habitat by the transport network constitutes one of the most serious global threats to existing biological diversity. [2]

Five main categories of adverse ecological effects due to the transport network are considerate as follows:

- Loss of habitat – this is the direct effect of roads construction leading to changing the physical surface along the route or the alteration of adjacent areas.
- The barrier effect – this is probably the biggest negative effect leading to the isolation of natural habitat areas.
- Victims of wildlife - mortality of the creatures is a frequent impact on wildlife trafficking.
- Disturbance and pollution - these two effects may cause the following undesirable effects :
    - hydrological changes - changing the natural topography of the land often leads to hydrological changes that will affect both wildlife and vegetation in those areas;
    - chemical pollution - a variety of pollutants resulting from traffic and from road surface causing groundwater, soil and vegetation pollution along the road route;
    - noise and vibration;
    - visual disturbances.
- Ecological functions of the verges - they can have negative consequences, by directing fauna in human populated areas, as well as positive consequences, by making a link between habitat areas.

3. WAYS TO COUNTERACT AND HARM REDUCTION

When planning new roads or improving existing ones to reduce habitat fragmentation the following principles are adopted: **ASSESSMENT, AVOIDANCE, MITIGATION, COMPENSATION, MONITORING.** [1]

ASSESSMENT OF VITAL AREAS

Habitat fragmentation should be minimized while still designing new facilities or rehabilitate the old ones. For this purpose Strategic Environmental Assessment (SEA) during the planning and programming as well as Environmental Impact Assessments (EIA) are undertaken. [1]

The main objective of the SEA and EIA is to identify possible negative environmental impacts before a decision is taken. Another objective of these evaluations is to ensure public consultation on the project that would be implemented.
Figure 1 shows a model of habitat fragmentation by roads network.

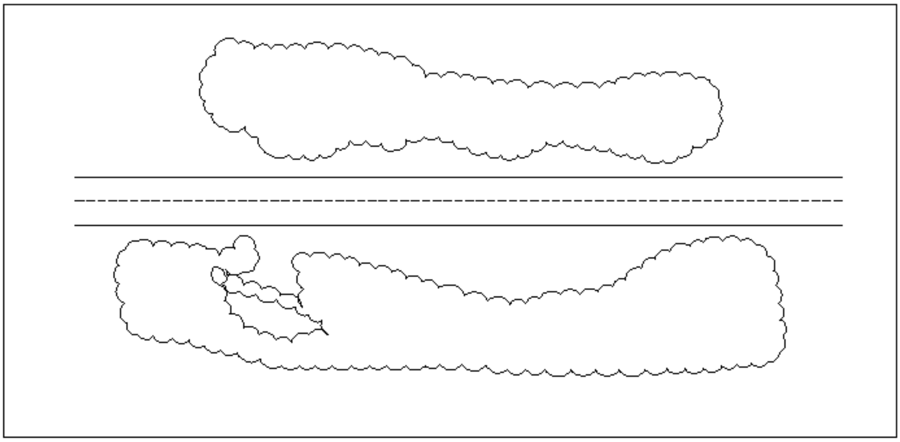

Fig.1 - Habitat fragmentation by road network.

AVOIDANCE

Adapting infrastructure (See Figure 2) to avoid crossing with natural habitats, reduction of land taken over roads or reducing disturbance of adjacent areas may significantly reduce these aspects but these measures do not avoid completly the fragmentation. [1]

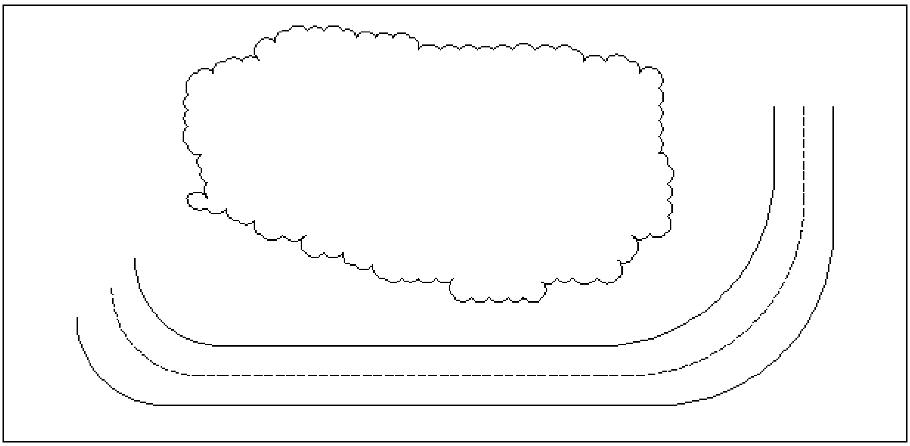

Fig 2 - Avoid of fragmentation.

MITIGATION

The barrier effect produced by transport infrastructure can be reduced by implementing solutions such as underground crossings which may leave undisturbed the surface of natural habitat or to adapt specific engineering work for facilitating local species crossings, which are called "green bridges"(Fig.3). [1]

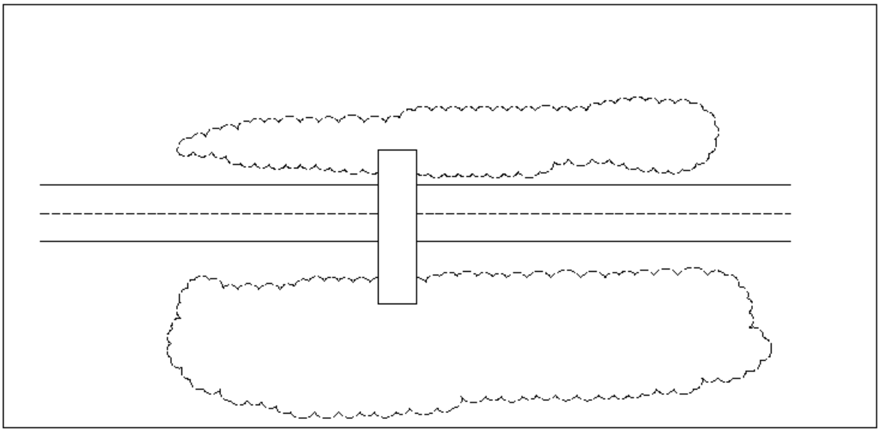

Fig.3 - Mitigating fragmentation by creating a "green bridge"

COMPENSATION

When fragmentation is unavoidable and mitigation measures taken are not sufficient to prevent habitat loss or degradation then compensation it is achieved in the form of creating a new habitat (see Figure 4). [1] Creating a new habitat is aimed for achieving a habitat quality close to that of existing and to offset the loss or degradation.

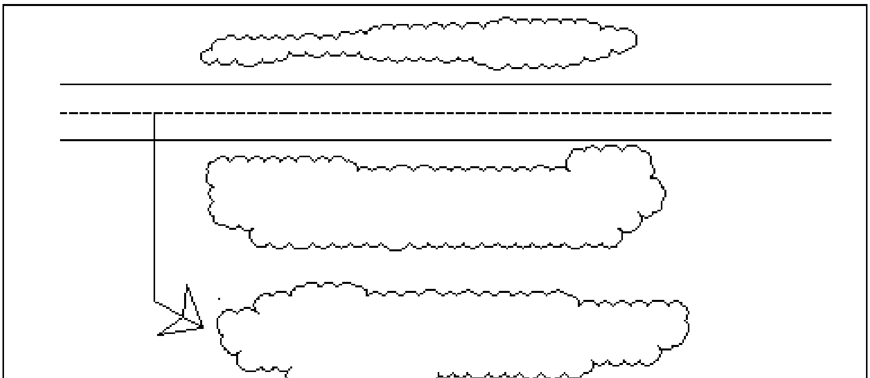

Fig.4 - Compensation for habitat loss by creating a new habitat.

MONITORING

For all these adopted methods such avoidance, mitigation and compensation must be properly conducted also the monitored in order to achieve the desired objectives. [1]

4. HABITAT FRAGMENTATION THROUGH TRANSPORT NETWORK.

Transportation infrastructure has developed over time in parallel with economic and technical development of human prosperity. However economic growth and social affluence resulting from the development of transport network can present disadvantages when negative impacts occur on the environment and human health.

According to SEA decision makers must be able to present different alternatives for mitigating the environmental impact. [3] In this aspect various research project such as **„Environmental Impact Assessment of transport infrastructure on natural and rural ecosystems**" [3] are still in the planning stage focus mainly on dealing with various impact indicators collected from field observations and with environmental impacts of transport infrastructure.

Data collection of geographic fragmentation , soil and water quality in areas adjacent to existing transportation infrastructure are harvested using the GIS program (Geographic Information System) and corresponding orthophotographic maps of chosen area for this study.

This project aims comparative investigation of representative geographical areas with a surface between 30,000 to 50,000 square kilometers [3] in different geographic areas e.g. Moldavia/Romania and Peloponnesus/Greece. The results of such investigations will be useful for improving assessing methods of the impact by transport infrastructure.

This proposal is an innovative approach that combines the use of environmental indicators, GIS software and a monitoring system to assess the impact of road network on habitat fragmentation, soil and water quality [4].

5. CONCLUSIONS

Although human activity began habitat fragmentation many centuries ago, the rapid increase in the density of transport network after the year 1900 led to accelerated fragmentation and accelerated negative impacts making them one of the biggest current global environmental problems.

Implementation of research actions at European level, like the Romanian-Greece project described above, could bring significant contributions to the reduction or prevention of habitat fragmentation by transport network**.**

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Exigencies and possibilities of expression in achievement of a university handbook

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Abstract

The paper discusses the exigencies and possibilities to improve the content as well as the general aspect and style of a technical handbook, dedicated to Bologna students in Civil Engineering.

Some usual weak points are revised. Then the technical writing recommendations that must be obeyed by authors are presented, in the particular case of books in the Mechanics of Deformable Bodies.

Key words: university handbook, technical, performance, Bologna

1. INTRODUCTION

The requirements of the fastest adaptation of the graduated students of a technical university of Romania in the year 2011, to the new conditions imposed by the permanently changing society, imply new criteria, performance indices and didactic technology, according to the expectations of graduates and employs.

If the accent of the teaching- learning process will be mainly laid on the individual study, on independent research and innovation activities, in order to assimilate knowledge and create practical skills specific to the field, specific tools will be needed, such as university handbooks available in the real or virtual libraries.

Publication which presents the content of the curriculum afferent to one or several university courses [8], the university handbook is mainly addressed to the students from a special field/specialization, in our case the Civil engineering.

The sudden transition from the restrictive, centralized, ideologically and economically planned publication system, to that of the initiative and free market, yet not always scientifically controllable, led to not always concurring expectations and results for more that a decade (1990-2000).

At present, under the conditions of alignment to the new legislation and the European Union exigencies, having quite different technical possibilities of information, publishing and dissemination or Internet presentation as compared to twenty years ago, the analysis of the context, results and prospective in this field are quite opportune.

2. EXIGENCIES AND ACCOMPLISHING MEANS

2.1. Addressability area

The university handbook dedicated to the 2011 student (with four years of study) need to be forming within the curriculum limits, at the same time developing the innovative and creative capacities.

The multiplication conditions some twenty years ago did not allow the publication of works in an attractive form, with explanatory drawings and high quality reproductions of special structures, structural systems, bridges etc.

Even if the author had the best intentions, these were annihilated by the finite product, an unaesthetic, unattractive book, printed locally on bad quality paper, which, unfortunately, contained the same amount of intelligence and work incorporated in it as today. In the future, for as long as they will still resist in libraries, the students will avoid them, both due to their aspect and the amount of information (hundreds of pages corresponding to courses extended on one or more years), and to their obsolescence, mainly caused by:

- the necessity to apply the norms and notations introduced by the Euro codes;
- The modification of the measuring units, utilization of other operating systems, of mathematical programming media and other new programming languages.

Here come the differences from both, an exhaustive treatise, addressed to the specialist educated according to the "old curriculum", and the university handbook elaborated up to some years ago, under the specified conditions.

2.2. Questions and answers

At present, for the first study cycle, having at disposal diversified technical means, it is possibly to conceive formative handbooks laying the accent on the practical side. The degree of complexity and author responsibility proportionally increase, being necessary to comply with the laws and norms imposed by the necessity to defend the copyright [2].

The integral multiplication of courses on external support favors the information dissemination among the students, but they prefer either the alternative of handbooks accompanied by CDs (DVDs) with self-evaluation tests and proposed problems [1], or the online alternative.

We write for a specific audience, consisting of students, researchers, specialists. Practically, this is a moment to re-analyze both the content and the forms of presentation and structuring of a much higher amount of information than that available two/ three decades ago.

Several questions appear now, such as: Is it necessary to demonstrate all the basic laws in the field? Do they benefit of enough space in a handbook destined to a new curriculum? What should we give up? Has the electronic material to be identical with the printed one? How to stimulate the interest and creativity? Is the form so important? Are the books of 20 years ago, still existing in libraries, still of interest for students? Is the finite product always updated, attractive and useful, or the old content is re-written and presented now in a more elegant package?

The answers are numerous and various, a more ample debate of the problem background being necessary in the future.

The objective of a proposed chapter, paragraph and exercise should be formulated as precisely and specifically, ideally in a sole phrase. For example, the aim of this set of exercises consists in *developing calculation skills for…*

In order to permanently check up the correctness of expression, specific working instruments are available, such as the on-line dictionaries etc. Yet, sometimes, a neutral eye (thorough, even if with another specialization) is especially useful to check the general structure and form of the presented material.

First of all, each material is to be impersonally re-read, seen from the position of the student or the reader who are looking for certain information within the corresponding pages. The information acquired after consulting the literature gets systematized, deciding the priorities. A much too exhaustive presentation will not bring the anticipated clearness.

Generally, the reader needs to be convinced of the relevance and significance of the information, the main ideas and procedures being highlighted in the context.

The attempt to use either a critical style or a much too descriptive one will make the text confuse and more difficult to understand. At the same time, enumerating the calculation relationships specific to our discipline, without explaining the physical phenomena or the main ideas and without underlying the results significance and the field of application will be of no use for a beginner who, most of the time, will mechanically reproduce the line of formulas and will identify the disciplines from the so/called "calculation" field with a mixture of formulas (the information being only remembered for a short time).

It is regrettable that in the new works the stress is not always laid on simple exercises, on signification of the symbols and some of the authors do not respect the European notations, which the reader will meet in practice all his career in specialized works and in technical norms.

In order to make clear and easier to understand the result of our work, one should take into account the following:

- realization of shorter sentences and paragraphs, highlighting the ideas being much more suggestive and useful, using rather enumerations than long sentences. The new ideas are to be highlighted accordingly, with the phenomenon description in simple words. Even if in the literature studied in the high school, the pupil came across with long sentences, having even to reproduce them to get a third, the effect of introducing such a long sentence in a technical text in quite unhappy. A short sentence should contain a single piece of information, avoiding the repeated utilization of the conjunction "and";
- pompous expression will also be avoided, since simpler sentences will be better understood and will call the reader's attention;
- the language specific to each branch of knowledge will be introduced step by step, being continuously updated according to the norms in force, yet avoiding the pedantry/flatulence.

There are unfortunate examples of persistence in using some obsolete denominations out of mimicry, inertia or lack of information. For example, one keeps using the name of "internal effort", also called "stress" (in Romanian) by geologists, instead of the correct name of stress resultant (axial force, shear, etc), as well as the symbols, which do not obey Eurocode exigencies.

A too scholastic elaboration in the cases of fields where the norms are more often modified (Metal Constructions, Reinforced Concrete, Construction Materials, and Technology) will constitute an obstacle in finishing a work addressed to students, teaching staff and specialists in the field. Here, the author has the possibility to bring corrections in successive electronic versions.

2.3. Means of expression and results

The specialty notions will be defined step by step, yet before they are introduced in a context. It will be aberrant to pretend redefining them whenever they are used. For example, need we explain the phenomenon of loss of stability and to define the Euler critical force whenever they appear in courses related to metal of wood constructions? Need we revert to the theory of tectonic plates and redefine the intensity of seismic actions in the handbooks dedicated to the structural design?

In all these cases, lists of symbols, abbreviations and proper names, with cross references to corresponding literature are recommended.

Yet, it happens that some doctoral dissertations or handbooks contains extended chapters, obviously with the character of bibliographical synthesis, which start from zero the presentation of Finite Elements Method, the principles of the Theory of probabilities, the above mentioned elements, etc.

Usually, the authors of works written in Romanian language make use of an impersonal style. Yet, this topic is widely discussed worldwide [5, 7], and, in research reports and works which describe experiments, the utilization of the personal style is recommended, most of times with the advantage of clearness and concision.

The sentence fluency can be checked by reading the manuscript as an impersonal reader. Any new idea or abstract concept can be efficiently introduced if we resort to simple examples, analogies and graphical elements.

We must underline the necessity to preserve the same denominations for a certain notion in the entire work (see the example of unit stress). At the same time, an abstract concept or a notion will not be replaced with a pronoun ("he, she, they", sometimes used to name a demonstration, a problem, a phenomenon, a structure, etc).

The introduction and the abstract are very important.

Usually, the first section of a chapter will be a short abstract of the corresponding chapter, as well as the text preceding the first division, etc.

Sometimes, the university handbooks start abruptly, eventually with a first refresher chapter. That is why it is important that each chapter has an introductory division, followed by the main content and, obviously, a short resuming subchapter, conclusions, perhaps tests and applications.

In the works with mainly applicative character, one can introduce a breviary at the beginning of each chapter, resuming the competences the reader can get after thoroughly running through this chapter [7].

The abstract can be descriptive or informative. The first one has a general character, without specific information or results. Usually, this type of abstract is drawn up before finishing the research.

The informative summary has a specific character; it can be found on the last page or on the book binding, with the presentation of the premises, case studies and material results; the text length is usually larger and it will be more complete, being specified in the specialized data bases depending on the work importance. The content of a chapter is also summarized, being useful for highlighting the ideas, presenting the guiding lines of the included material, as it has been mentioned previously.

2.4. Presentation of the material

Every author wishes to present his material in a more attractive form. The conclusions from the specific field of the *Mechanics of deformable bodies* will be easier to draw if the results are graphically presented, while in other descriptive areas the tables are preferred.

The photos from the author’s portfolio or those which comply with the intellectual property can confer a new and remarkable opening of the reader’s understanding. In the on-line courses, the insertion of applets and video-files is opportune.

In order to support the reasoning, to check and pin down the knowledge, figures and tables are necessary. Nevertheless, the work should reach its target without using in excess the graphical or decorative elements.

The *font* and letter size are established at the beginning of drawing up. Generally, a certain font (Times New Roman) is recommended, as well as 11 or 12 pt for text, but one should also test the reader’s options. In certain cases, for the paragraphs suggested as additional readings, the font will be diminished.

The chapters will be divided in subchapters which, in turn, will be divided in sub-divisions once. An excessive numbering will tire the reader and result in loosing the main ideas. If and additional sub-division is necessary, this will be carried out by means specific to desktop publishing (lists, dots, etc). Each of these divisions/subdivisions represents a unit. It is essential that each of these subdivisions has a unitary content, indicated by the title.

At the same time, there have to be at least two units of comparable length at the division level. It is not mandatory that all the chapters are divided in sub-chapters or sub-subchapters.

The paragraph must contain a single coherent idea and its length does should not exceed half a page. On the other side, if the ideas structuring requires a small paragraph length, a list of numbered problems is preferred to a succession of short paragraphs.

2.5. Symbols and equations

There are three basic rules [7] essential to provide the clearness of a text in which the mathematical symbols and calculation relationships are frequently used:

- note the variables with *italics*, to distinguish them from the text characters, maintaining the same symbol when writing the equations. The matrices are written with **bold** letters. Adopt a unique name for a certain physical parameter. For example, modulus of elasticity, with the symbol E, will be named the *Young* modulus only in Physics or in English language. Sometimes, the confusion comes from using the same symbol for several parameters. Thus, in the elastic theory, the symbol ν was preferred for Poisson's coefficients, since the symbol μ also appears in other fields (Reinforced concrete). The symbol ? appears in several domains of the Engineering (Stability, Thermal analysis, Calculus). Here comes the importance of the lists with symbols and proper names attached to the handbook;
- do not start a sentence with mathematical symbols; this will result in increased ambiguity and understanding effort. A sentence like:"*x*" represents…" will be replaced with "the variable *x* represents…"
- write the equations in separate lines and number them.

Nevertheless, the haste and negligence make us forget to comply with these rules. Taking these rules into account will lead to a better understanding of the teaching materials destined to students, master students, doctor students, as well as to the specialists in the field.

3. CONCLUSIONS

The ability to draw up a university handbook from the technical field can be improved taking into account simple, clear rules, different from those specific to writing a fiction book.

Any handbook can be improved in successive stages [4], following a short breviary that can be resumed as follows:

- use the clearest possible formulation of the proposed purpose, such that the entire material is subordinated to it;
- use a simple language, with short sentences and paragraphs, avoiding wordiness and pompous expression;
- avoid to repeat ideas or words;
- use diagrams, graphs, figures, etc which facilitate the clearest understanding of the course/handbook;

- use additional explanations (when necessary) related to certain equations, formulas, norms, etc.

And last but not least, use good quality paper, print in the most attractive graphical conditions for those who use these materials, preserving an attractive format in the on-line version too.

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Evaluation of technical condition of road pavements and applications of an advanced methodology for prioritization of maintenance works

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Summary

This paper presents an advanced methodology for the investigation of road pavement condition and the use of a specific software for prioritization of the road works. Finally a case study conducted the county road DJ 282B presented.

KEYWORDS: Pavement Condition Index-PCI, Pavement Serviceability Index-PSI, road distress, strategies for intervention, prioritization, of road works

1. INTRODUCTION

With the passage of time the highway pavements deteriorate even if good quality materials and advanced construction techniques are used. The pavement structures are designed to deteriorate in time, depending on: their strength, traffic volumes and axle loads, environmental conditions and the maintenance policy. It means that pavements are the only engineering structures which are designed for failure after, a number of years in service. Thus, it is possible to classify the pavement failure in two categories: structural and functional failure. In the structural failure the bearing capacity of the pavement is compromised and in the functional failure the deterioration refers to the reduced ability of the pavement surface to provide smooth, safe and economic ride to the users.

The deterioration modes can be divided into load and non-load related. The traffic on the pavement structures generates load associated causes. The non-load associated causes are divided into environmental conditions, construction quality and special problems.

In the deterioration mechanism a particular combination of factors is involved. Figure 1 presents a specific deterioration curve, as function of the evolution of pavement quality, in time.

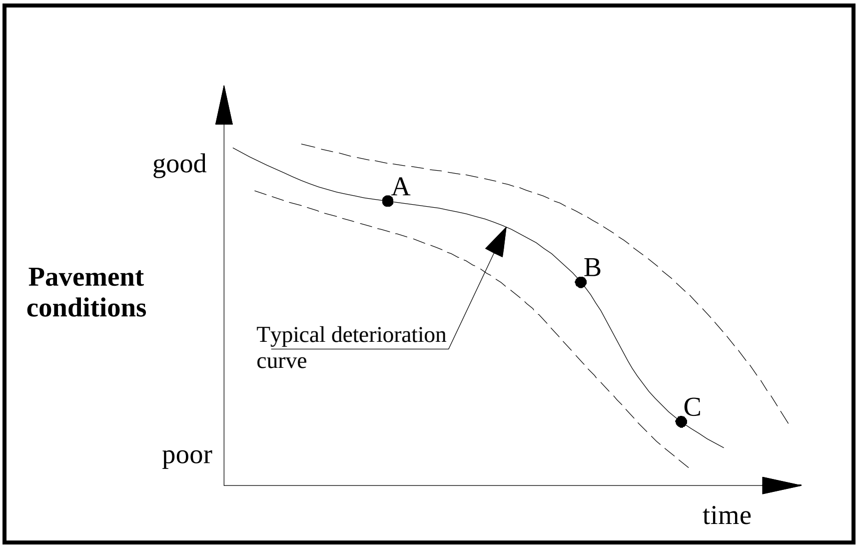

Figure 1.Typical pavement deterioration curves

This continuous line is considered to be a typical distress curve. In relation with Figure 1, in the early stage of the pavement life the pavement condition suffers a fast and small drop, until the point A, afterwards the deterioration rate slowly increases during the most of its life, reaching the point B and after this point the deterioration rate increases faster so that the pavement condition is reduced quickly in a short period of time. When the point C is reached, the pavement is considered failed the road because impracticable and after this point the deterioration rate is decreasing again very fast.

Because of the parameters involved, it is not possible to determine a unique deterioration curve valid for all pavement sections but there are several features that can be generalized for the majority of the highway pavements.

2. IVESTIGATION OF ROAD PAVEMENT CONDITION

2.1. Investigation of pavement distresses

According to the international practice [2], for flexible pavements, a number of nineteen specific types of distresses have been considered, as shown in Table 1.

Table 1.The main types of specific distresses for flexible pavements according to the literature [2]

Cod	Type of Distre
#1	Aligator cracking
#2	Bleeding
#3	Transverse and block craking
#4	Bumbs and sags
#5	Corrugations
#6	Depressions
#7	Edge cracking
#8	Joint reflectioncracking
#9	Shoulder drop off
#10	Longitudinal and transverse cracking longitudinal joint cracking wheel cracking
#11	Patching-utility cuts
#12	Polis aggregates
#13	Potholes
#14	Railorod crossing
#15	Rutting
#16	Shoving
#17	Slippage cracking
#18	Swell
#19	Wethering and raveling

2.2. Investigation of the existing pavement structure

For the scope of this study a road sector from Km 50+150 to Km 51+180 was selected on the county road DJ282B, having the pavement structure as shown in Figure 2.

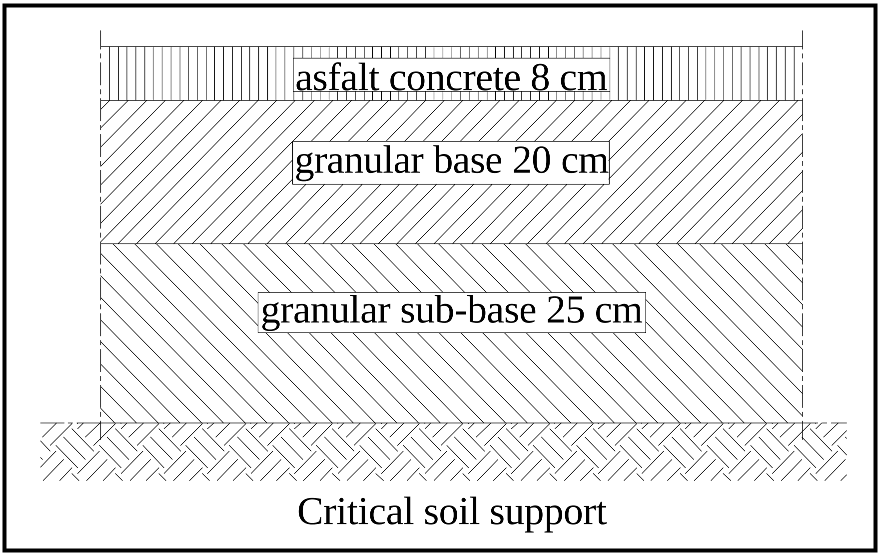

Figure 2. Road pavement structure on county road of DJ282B – Flamânzi, Botosani

2.3. Evaluation of the road traffic

The calculation of the design traffic has been based on the figures recorded during 2010. Traffic Census, expressed in average daily traffic ADD, resulting a total number of 2100 phyzical vehicles.

2.4. The bearing capacity of the sub-grade soil

The bearing capacity of the sub-grade soil, classified silt clay, has been evaluated in terms of critical soil support value expressed, by a Resilient Modulus-MR, calculated with the relation (1):

$$MR=1500*CBR \quad (1)$$

where:

MR – resilient modulus of the subgrade soil

CBR – Californian Bearing Ratio, specific for this type of soil [3]

3. STRATEGIES OF INTERVENTION USED IN ROAD MAINTENANCE & REHABILITATION OF FLEXIBLE PAVEMENTS

A number of thirty strategies of intervention, developed according to the existing existing literature [2] have been selected as shown in Table 2.

Table 2. The specific strategies of intervention selected for flexible pavements according to the existing practice [2].

Strategy Code #	Description	Category
0	No action required	
1	Do-nothing	
2	Seal cracks	
3	Patching critical areas	
4	Extensive patching	
5	Surface treatment loc. Area	routine
6	Milling/text. loc. Area	maintenance
7	Premix leveling loc. area	
8	Skin patching	
9	Fog seal	
10	Surface treatment	
11	Double surface treatment	
12	Surface treatment chips	
13	Double surface treat. chips	
14	Slurry seal	
15	Hot aggregate spray	
16	Texturize surface	
17	Open friction courses	
18	Thin overlay ¾”	
19	Crack reflection treatment	major
20	Structural overlay	maintenance
21	Heat/planer	
22	Heat/planer (+ rej. /mat.)	
23	Heat/planer (+ 1” AC)	
24	Surface milling (max. 1”)	
25	Hot milling (max. 1”)	
26	Leveling course	
27	In-place recycling cold	
28	Plant mix recycling hot	
29	Reconstruction	

4. PRIORITIZATION OF ROAD WORKS BY USING SPECIALIZED SOFTWARE NAMELY: PAVEMENT REHABILITATION ANALYSIS SYSTEM [2]

A micro-computer program has been used to determine the different pavement indicators and to display the set of maintenance and rehabilitation alternatives for this road sector. This software has been developed in order to use minimum available input information required in a special form in order to facilitate the data collection and to make the program friendly user. As output, the program generates a list of feasible alternatives in function of the input data.

After all the variables and the parameters are entered, the program starts the execution, prints the most important variables and gives the possibility to review them. The program provides for each of the various alternatives the expected live and the specific economic indicators.

Thus, a typical output has three parts. The first part includes the general project information like project name and dates. The second part contains the project variables and indicators and all input data to facilitate a future technical and economical use and interpretation of results. The third part has the results of the analyses which include a list of feasible alternatives selected by the software and the expected life and economic analysis corresponding to each alternative.

Here follows an example of application of this software to the road sector mentioned above:

A) Input Data:

- Project description: The County Road Flamânzi, DJ282B Km 50+150 – Km 51+180.

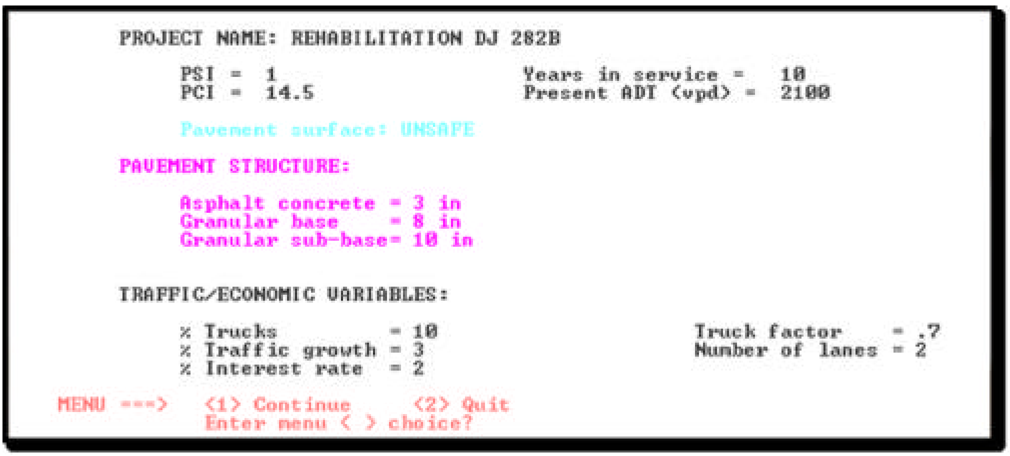

Figure 3. Print screen with the main input data.

- Distresses recorded in the road sector are shown in the Figure 4.

PCI - DEDUCT VALUES

#	DISTRESS	DEDUCT VALUE
1	Alligator cracking	5
3	Block cracking	10
4	Bumps and sags	5
5	Corrugations	5
6	Depressions	10
8	Jt. reflex. cracking	3
10	Long/trans. cracking	20
11	Patching/utility cut	5
13	Potholes	7
15	Rutting	5
17	Slippage cracking	3
18	Swell	3
19	Weathering/raveling	5

Is this correct y/n:?

Figure 4. Print screen with distresses investigated in the road sector DJ282B.

B) Output data

Based on the known data as shown in Figure 3 & Figure 4, after running the software a number of ten alternatives for the rehabilitation of this road have been proposed, as shown in Figure 5.

PROJECT NAME: REHABILITATION DJ 282B

ALT No.	DESCRIPTION		<LIFE (yrs)> TOTAL	ADDED	COST $/LA-MI	AVG.ANN. COST
	CURRENT PAVEMENT CONDITION		0.0			
1	Leveling loc.area	(+)				
	Thin overlay 3/4 in.		0.4	0.4	15339	38882
2	Leveling loc.area	(+)				
	Structural overlay **		2.5	2.5	29208	12094
3	Heat/planer (+1 in AC)		1.7	1.7	18304	11058
4	Leveling course	(+)				
	Thin overlay 3/4 in.		0.6	0.5	26963	54734
5	Leveling course	(+)				
	Structural overlay **		4.0	4.0	40832	10723
6	Plant mix recyc. hot		2.7	2.7	33792	12981
7	In-place recyc. cold	(+)				
	Surface treatment		0.3	0.2	26752	135361
8	In-place recyc. cold	(+)				
	Thin overlay 3/4 in.		0.7	0.6	33299	56385
9	In-place recyc. cold	(+)				
	Structural overlay **		5.3	5.3	47168	9468
10	Plant mix recyc. hot		2.7	2.7	33792	12981

** Overlay thickness (in) = 2

Figure 5. Final print screen with the feasible alternatives proposed by the software.

5. CONCLUSIONS

Considering the output results shown in Figure 5, we may observe that from the proposed ten alternatives for intervention, the cheapest one (ALT. No. 1/ Cost 15,339 $/LA-MI) consisting in leveling the surface in local areas with thin overlay (3/4 in) extending the actual life of the pavement with 0.4 years.

In the same time a more consistent strategy, such as ALT. No. 2 (Leveling local areas by heat planner +1 in AC) is extends the existing life of the pavement with 2.5 years.

A similar strategy but realizing the leveling with a structural overlay of 2 in asphalt layer is extending the existing life with four years, but at a higher cost (40,832 $/LA-MI instead of 29,208 $/LA-MI).

Finally, we may observe that a more radical strategy (ALT. No. 9/ in place recycling cold and structural overlay of 2 in) is extending the actual life with five years, at a specific cost of 47,168 $/LA-MI.

We may conclude that the most economic alternative is the last one, even if the initial investment is higher than the cost of the other alternatives, because the annual average maintenance cost for all these five years is the lowest one.

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Iasi, România, December 10, 2010

Asphalt mixture reinforced with cellulose fibers for a road pavement

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Summary

The results of the research accomplished at the Road Laboratory of the Faculty of Civil Engineering of Iasi for establishing the compositions of the MASF16 asphaltic mixtures reinforced with cellulose fibres are presented. This particular asphaltic mixture was used for the road pavement over a bridge in the county of Bacau . After the establishing of the optimum bitumen content: 5.9% and of fibres: 0.6%, the rutting test was applied to the mixture. The experimental results were in conformity with the requirements of the Romanian standard SR 174/1-2002.

KEYWORDS: asphalt pavement, asphaltic mixture, cellulose fibers, reinforced.

1. INTRODUCTION

The reduced tensile strength from asphalt mixtures layers leads to cracks and ruts in the road surface which are subjected to alternating applications under the action of heavy traffic. Since the mid-60s, to combat these effects began using dispersed reinforcement of asphalt mixtures of hard bituminous layers of natural organic fibres(cellulose, steel, etc..) or artificial (poliester, polypropylene, etc). [1...3].

These asphalt mixes are produced by incorporating in the mixture mass a certain amount of staple fibres.

The addition of fibres has a double effect:

- In short term, during preparation, transport and commissioning work the fibres’ allow the increase of the binder amount and avoid the leaks and the segregation.

- In long term, in correlation with increasing binder content improves the performance of asphalt mixture providing a better fatigue behaviour [2,3].

Studies have revealed that certain types of fiber (cellulose fiber) increases cohesion of bituminous mastic for manufacturing temperatures but do not lead to significant hardening of the mixture at operating temperatures. They play a vital role in increasing the elastic modulus known in terms of mixtures performance.

2. LABORATORY STUDIES

In the Road Laboratory of Technical University of Iasi was established the composition of asphalt mixtures type Asphalt Mixture Stabilized with fibres (MASF 16) to achieve the wearing course for two bridges located in the county of Bacau.

The crusher sand and the chippings were supplied from the career Suseni-Chileni (county of Harghita), the filler from the mining Delnita (Harghita County) and the bitumen from OMV HUNGARIA MINERALOEL GMBH.

Grading curves of natural aggregates used are presented in Table 1 together with the composition of the mixture of natural aggregates and its grading curve.

Filler characteristics are presented in Table 2 and the main characteristics of bitumen, used in the preparation of the mixture, in Table 3.

With these materials were made test samples, with Marshall cylinders (? = 101,6 mm ; H = 63,5 mm) prepared from asphalt mixtures with 5% of bitumen and cellulose fibre ITERFIBRA/C of 0.6% from the weight of the mixture (Manufacturer ITERCHIMICA –Italia recommended values are between 0,3 % and 1,0 % from the weight of the mixture).

Table 1 – Aggregate grading curves of the mixture of natural and crushed aggregates

crt. no	Characteristic	Chipping gas 8-16	Chippings 4-8	Crushed sand 0-4	Filer	Combination %	Limits SR 174/1-02
1.	Granularity :						
	- passed through sieve 25 mm, %	100,00	-	-	-	100,00	100
	,, 16 mm, %	95,89	100,00	-	-	97,82	90...100
	,, 8 mm, %	3,99	89,78	100,00	-	46,96	44...59
	,, 4 mm, %	0,45	16,38	98,45	-	29,48	25...37
	,, 2 mm, %	-	2,98	67,05	-	22,35	20...25
	,, 1 mm, %	-	-	42,76	-	18,56	16...22
	,, 0,63 mm,%	-	-	32,22	100,00	17,19	13...20
	,, 0,2 mm,%	-	-	17,46	99,10	15,15	11...25
	,, 0,1 mm,%	-	-	10,17	88,40	12,81	10...14
2.	Composition MASF 16, %	53,00	21,00	13,00	13,00	100,00	-

Table 2 – Filler features of Delnita

Feature	Unit	Values	Limits STAS 539-79 (AND 539-02)
1. Grading : - passed through sieve: 0,63 mm - passed through sieve: 0,2 mm - passed through sieve: 0,125 mm - passed through sieve: 0,100 mm - passed through sieve: 0,071 mm - passed through sieve: 0,063 mm - passed through sieve: 0,020 mm	 % % % % % % %	 100,00 99,10 94,20 88,40 77,20 73,10 22,03	 100 Min. 98 - - Min. 72 - (Min. 20)
2. Humidity	%	0,41	Max.2
3. Hydrophilic factor	%	0,86	Max.1
4. Apparent density after settling in benzene	g/cm^3	0,64	0,5 ... 0,8
5. Voids Coefficient compacted state	-	0,39	0,3 ... 0,5

Table 3 – The characteristics of bitumen

Current issue.	Characteristic	Unit	Values	Limits SR 754-99
1.	Penetration at + 25^0C	1/10 mm	66	66...80
2.	Softening point (I.B.)	^{0}C	49,1	49...55
3.	Penetration Index: Susceptibility index	I.P. “a”	- 0,76 0,45	Structure sol-gel

Physical and mechanical values are presented in Table 4.

Based on the analysis of these values the optimal bitumen content of 5.90%, by weight mixture has been established. From a set of sets of plates (30,5 x 30,5 x 3,5 cm) have been prepared in order to be tested for rutting in the Central Laboratory of Roads and Bridges Regional Directorate Iasi. Testing set consisted of 6 plates.

Te following values have been determined:

- Resistance to permanent deformation;
- Modulus of elasticity and permanent deformation at + 15^0 C ;

As presented in Table 5.

Table 4 – The values of physical and mechanical mixtures prepared in the laboratory

Curent Issue.	Bitumen, % of mixture weight	Apparent density (kg/m^3)	Water absorption (% vol)	Marshall test		
				S, kN	I, mm	S/I, kN/mm
1.	5.3	2.345	3,759	8,2	2,55	3,216
2.	5.6	2.352	3,187	8,8	2,61	3,372
3.	5.9	2.357	2,533	9,1	2,73	3,333
4.	6.2	2.353	1,857	8,7	3,22	2,702
5.	6.5	2.345	1,408	7,9	3,63	2,176

Table 5 – Rutting test results

Current Issue	Characteristic	U.M.	Value	Limits SR l74/1-2002
1.	Resistance to permanent deformation : - dynamic creep at+ 400C si 1800 pulse ; -strain rate at + 600C (V.D.O.P.)*; -rut depth.	 mm mm/h mm	 0,281 0,48 5,36	 Max. 1,0 Max. 8,0 Max. 9,0
2.	Elasticity module + 150C	MPa	6036	Min. 4000
3.	Constant fatigue deflection (3600 pulse) at + 150C	mm	0,732	Max. 1,0

* Strain rate.

3. Experiments in the current path

In September 2008 , asphalt mixes were laid on the roadway bridge over the river Trotus on the DN12A, at km 70+235 and km 70+820. Physico-mechanical characteristics determined on the samples collected from asphalt mixtures are presented in table 6.

Table 1 – Aggregate grading curves of the mixture of natural and crushed aggregates

Feature.	Characteristic	Unit	Sample 1	Sample 2	Values allowed
			DN12A-Kilometer 70+235	DN12A-Kilometer 70+820	
1.	Bitumen mixture content	%	5,93	6,11	6,5...7,5
2.	Grading of the mixture of natural aggregates:	%			
	-trec - passed through sieve 25 mm, %		100,00	100,00	100,00
	- passed through sieve 16 mm, %		97,23	95,37	90-100
	- passed through sieve 8 mm, %		46,45	45,70	44-59
	- passed through sieve 4 mm, %		30,07	28,44	25-37
	- passed through sieve 2 mm, %		22,71	23,85	20-25
	- passed through sieve 1 mm, %		19,11	20,76	16-22
	- passed through sieve 0,63 mm,%		18,36	19,03	13-20
	- passed through sieve 0,2 mm,%		13,92	14,85	11-15
	- passed through sieve 0,1 mm,%		12,98	13,19	10-14
3.	Schellenberg Test	%	0,14	0,10	0,2
4.	Apparent density on Marshall cylinders	Kg/m^3	2.368	2.419	2.300
5.	Water absorption on Marshall cylinders	%	1,034	0,318	-
6.	Marshall Stability	KN	9,0	8,8	7,0
7.	Marshall Flow Index	mm	2,92	3,16	1,5-3,5
8.	Marshall Stiffness mode	kN/mm	3,08	2,86	-

4. CONCLUSIONS

Laboratory studies have led to the production of asphalt mixture compositions stabilized with cellulose fiber, MASF 16 type whose characteristics have values that fall within the limits imposed by standard : SR 174/1-2002 although the optimal dosage of bitumen - 5,9% is lower than the recommended minimum value of 6,5%. Tracking behavior of pavements made from reinforced asphalt in September 2008 will identify the benefits brought by the addition of cellulose fibers.

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About bridge engineering in Ireland. Case study.

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Summary

In this paper will present a short description about National Roads Authority and is shown that an important function is the management of infrastructure but also in road development in Ireland transporter.

I present below some information about the database Eirspan, when it was founded, which is the purpose of these databases, what purpose serves and how many bridges have in administration..

Is shown that most bridges were built in Europe over the past 50 years, which also means there is a large number of bridges that have to cope with current traffic.

Today, percentage shows as the most widely material used in bridges engineering is concrete, even though many have many advantages and also disadvantages.

According to the literature, is show when the concrete was first used and is give examples of bridges like concrete, stone and steel.

KEYWORDS: concrete bridge, stone bridge, iron bridge.

1. INTRODUCTION

The transport network is extremely important to Europe's economic and social development. It has been a crucial factor in economic growth and prosperity and it plays an important role in the everyday life of the citizens of Europe, by allowing the quick, easy and safe movement of people and goods.

NRA National Roads Authority is to secure the provision of a safe and efficient network of national roads. For this purpose, the Authority has overall responsibility for planning and supervision of construction and maintenance works on these roads. Authority includes any training, research and testing activities necessary for the execution of its functions, which include project planning, construction, maintenance and operations. The National Roads Network comprises 2739 kilometres designated as National Primary and 2676 kilometres designated as National Secondary.

IABMAS- International Association for Bridge Maintenance and Safety is summarised in the report the following inventory information: the total number in the system of bridges is 2800.

In 2001 was made the Eirspan (data base for Bridge Management System) was introduced to coordinate and integrate activities such as inspections, repairs and rehabilitation work to ensure optimal management of the national road structure stock.

Fig. 1. Road Map of Ireland

2. BRIDGES GENERALLY

Most bridges on the national road networks in the European Union have been built during the last 50 years although some are much older. However the increasing volume of traffic and maximum weights of individual vehicles mean that for many structures the loads to which bridges are being subjected are far higher than those envisaged when they were designed.

Maintaining structures in a serviceable condition is complicated by the wide variety of structural types. Whilst the majority of modern structures are of reinforced or prestressed concrete construction, there are also a large number of composite bridges with steel beams supporting a concrete deck and a smaller number of steel bridges. The majority of the older structures are of masonry arch construction. Each type of structure behaves differently, suffers from different types of deterioration and has different maintenance needs. All this adds to the difficulty of ensuring that bridges are properly maintained.

The use of concrete in Ireland can be traced as far back as 1850, in the foundations of a bridge crossing the River Glyde, in Dundalk. As the benefits of this material were realized, concrete was increasingly used as a solution to construction problems.

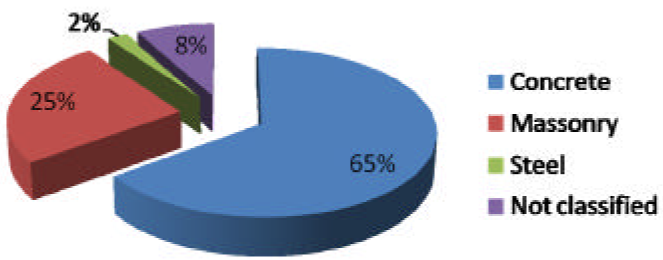

Fig. 2. Percentage of bridge with constructions materials

2.1. Concrete Bridge

Concrete dominates the materiality of Irish bridges. Most modern bridges in Ireland (approximately 98%) are constructed from concrete – reinforced concrete, precast pre-tensioned concrete and post-tensioned concrete are all commonly used. Cement is natively produced and is more economical to use in constructions. A major fraction of the Irish landmass sits on top of limestone bedrock making clinker and aggregate readily available for cement and concrete production.

Since over 90% of all bridges in Ireland are short span, a large part of the practice up to recent years has centred on the above three types.

The ability of concrete to take on virtually any shape to be in effect moulded permits organic and sculptural design.

Piers provide the most striking examples of freedom with the materiality of concrete: the variety and expression to be found on motorway overbridges clearly exhibit this possibility. In response to the brutality who would have us build dumb expressionless rectangular walls it is remembering that piers make up something like only 7% of the cost of a bridge.

Figure 3. Killarney road bridge piers

Figure 4. Kildare bridge

2.2. Stone Bridges

The large majority of bridges in demesnes are built of locally available materials, such as limestone, granite or other naturally occurring stone. The largest number of masonry bridges are to be found in locations were it was necessary for the driveway to the house or castle to cross a significant body of water, mostly rivers, but occasionally an artificial lake or other watercourse.

Figure 5. Queen Maeve Bridge, Dublin

This is the oldest standing bridge spanning the Liffey river. Built in 1764 and completed in 1768, it was originally known as Queens Bridge after Charlotte of Mecklenburg, wife of George III. It was renamed after the legendary Queen Maeve of Connaught who invaded Ulster. The bridge replaced an earlier structure named Bridewell Bridge built in 1683. Was designed by Charles Valency, design elliptical arch stone bridge, with 3 span, total length is 43m and width is 10,2 m.

2.3. Iron Bridge

Compared with the use of stone, iron was not used to any great extent in Ireland for demesne bridges. However, there are a small number of relatively important demesne bridges constructed of cast- and wrought-iron that were erected by the landed gentry for their decorative qualities, some imported, some fabricated in Ireland. Types of iron bridges include beam, truss, arch and suspension.

Rory O'More Bridge is a road bridge spanning the River Liffey in Dublin, Ireland and joining Watling Street (by the Guinness grounds) to Ellis Street and the north quays.

Figure 6. Rory O’More Bridge, Dublin

The first bridge by wood on this site, built in 1670. The timber bridge was replaced by a stone bridge in 1704, which was replaced in turn by the present day structure. Designed by George Halpin, the bridge was fabricated at the foundry of Robert Daglish in St Helens, Lancashire, from cast iron (with a wrought iron deck) and is supported on granite abutments. The bridge was renamed in 1939 for Rory O'More, one of the key figures from the plot to capture Dublin as part of the Irish Rebellion of 1641.

Figure 7. James Joyce Bridge, Dublin

Named after the Dublin author James Joyce is set in a house facing the bridge, crosses The Liffey river. This was projects by Santiago Calatrava for Dublin. Costing just under €9 million and the steelwork was manufactured by Harland and Wolff in Belfast. It is a single-span structural steel design, 40 m long, with the deck supported from two outward angled arches. Carriageway width is 13-18 m and sidewalk width is 3-6 m.

Figure 8. Samuel Beckett bridge, Dublin

The 120-metre long, 48-metre high bridge spans the river Liffey from Sir John Rogerson's Quay near Macken Street on the south side to Guild Street at the site of the new National Convention Centre on the north side.

The shape of the spar and its cables is said to evoke an image of a harp lying on its edge. The total cost of the project is estimated at just under €60m, which was also including a major upgrade of the approach roads.

The bridge will be capable of rotating through an angle of 90 degrees to facilitate maritime traffic.

3. CONCLUSIONS

In Ireland the nature of existing and new bridges is very varied with many masonry arch bridges dating from the 17th and 18th centuries still in service in parallel with newer steel and concrete bridges.

As shown in this some examples give of bridges, Ireland has an important number bridges structures as use, like remark they hold a exceptional design, quality execution and good policy for the maintenance of bridges structures.

Made of concrete bridges occupy a growing proportion compared to other materials such as wood, stone or steel, but is the most important factor is the cost of construction is lower when these materials have a local origin.

In literature found bridges designers use concrete is becoming more and more, this is happening in the entire World.

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Analysis of the Uneven Settlement Phenomena at the CL 17 Building Site, Zugravi Area, Iasi City

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Summary

The paper main objective is to emphasizing the causes of uneven settlements which took place on a building site with a structure of lamellar frames, with height regime S+P+7E.

In the paper is being analyzed the setting up and manifestation of uneven, unstabilized settlement phenomena.

Also, are presented the results obtained through the application of several remedial measures, with the purpose of stopping the progressive settlements and reestablishment of construction functionality.

The paper points out the influence of humidity variation on the Bahlui clay resistance to strength, for different conditions of pressure state in soil.

KEY WORDS: foundation soil, soil stratification, footing pressure state, reinforced concrete structure.

1. GENERAL CONSIDERATIONS

The present paper aims at emphasizing a special phenomenon which took place in July 1986 at the CL 17 building in Zugravi area, City of Iasi. As a consequence of the investigations realized by a group of researchers from Gh. Asachi Technical University, Faculty of Civil Engineering and Building Services, Iasi and Civil Engineering Technical University, Bucuresti have been determined the important and generalized causes of the uneven settlement phenomena, recorded starting with 2nd of July 1986.

Is being mentioned a series of remedial measures for the destructions with the purpose of stopping the progressive settlements and reestablishing the construction functionality.

2. CONSTRUCTIVE ELEMENTS AND FOOTING SOIL CHARACTERISATION

Adapted after a type project, the CL 17 building has the following constitution: technical basement, ground floor and 7-8 floors.

The resistance structure is constructed with lamellar frames with pillars of monolithic reinforced concrete, girders, floors and front panels, prefabricated, from concrete C8/10.

The infrastructure is composed of a basement from monolithic reinforced concrete, in the form of a rigid box, with longitudinal and transversal walls which rest on simple concrete bed, with the width of 1,70 m. The floor over the basement is realized with monolithic reinforced concrete. The top and the bottom of the construction walls are provided with belts of reinforced concrete.

In figure 1 is presented the building zone framing as well as its site during the phenomena manifestation.

Figure 1. Building zone framing and its site during the phenomena manifestation

In order to identify the site and to characterize the stratification have been realized several drillings. The drillings profiles pointed out a non homogeneous stratification composed from a fat clay, with grey-black color, in plastic dense state with variable thickness (2,0-3,0 m), in the site area. Under the fat clay was found a layer of silt clay of 0,80-1,00 m thickness and towards the base a sand with gravel, with the thickness between 2 and 5,50 m.

The building foundation is situated on fat clay, in this same layer being also situated the hydrostatic level.

Following the investigations realized in the field with a thickness of 2,00-3,00 m has been observed the water presence, situated and accumulated in permeable bags, at depth of 1,50-3,00 m.

The aggregate of natural alluvial formation field is situated on a thick layer of compact marl clay.

3. THE PHENOMENA APPEARANCE AND MANIFESTATION

The starting moment of the first uneven, unstabilized supplementary settlement, took place during the finishing operations by the engineer.

Starting with 2nd July 1986 has been observed an uneven supplementary settlement of the building, with a high developing speed, evaluated through monitoring, at about 5 mm/hour.

In the building basement have been observed cracks and displacements in the floor due to repression of soil bellow footings. All the rooms from the basement presented a water layer with variable thickness of 10-30 cm.

Figure 2. Displacement and shifting of floor from the basement

As a consequence to important uneven supplementary settlement, with progressive character, the building leaned dominantly, towards the street but also transversal direction, towards the neighbor building, CL 16, as can be seen in the following images.

The affected constructive elements are:

- To exterior, on the building outline has been signalized the presence of a continuous crack;
- The soil under foundations repressed in the basement, in all the rooms;
- As a consequence of soil pressure, the floor fractured;
- In certain rooms the floor fragments have been dislocated and displaced, by movement, reaching the bottom part of the floor over the basement, as can be seen in figure 2.

4. RESEARCH ON THE UNEVEN SUPPLEMENTARY SETTLEMENT

Following the establishment of important uneven supplementary settlement, by installing certain topographic landmarks in the construction base, took place the daily recording of vertical displacements and building leaning.

On the construction perimeter, have been realized drillings in order to verify the stratification and static penetration in order to establish the thickness variation and physical state of fat clay.

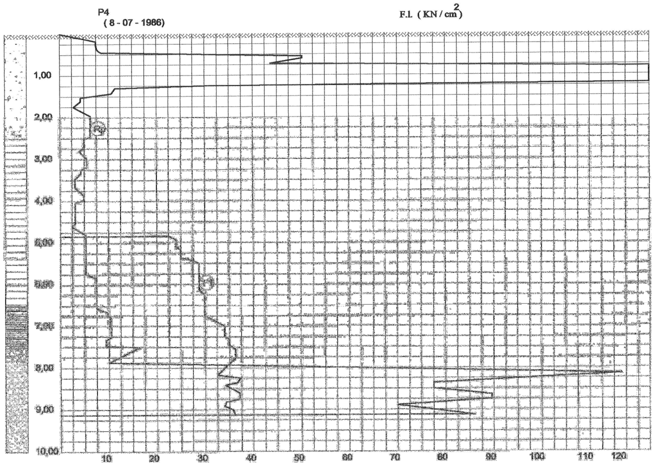

Figure 3. Penetrometer diagrams for resistance on top and on the mantle

Simultaneous, on the construction corner, as opposed to the leaning direction has been realized at the exterior an open borehole.

In the open borehole, the vertical wall digging under the footing base, has failed laterally on approximately 0,50m height. The soil mass displaced bellow foundations is presented under the form of fragments ensemble, with transversal dimensions of about 2,0-4,0 cm, with shiny faces, uneven failures resulted from the breakage of soil by shear strength.

In figure 4 is presented the aspect of the clay fragments from the displaced area due to soil lateral repression and a frontal view of the soil portion with lateral failure.

Figure 4. Frontal view of the soil part with lateral failure and clay fragments from the dislocated area.

5. MEASURES FOR THE STOPPING OF THE UNEVEN SETTLEMENT PROGRESSIVE PROCESS

The prompt measure to stop the uneven progressive settlement process, which threatened by its uncontrolled development, to produce structure stresses higher than the ones for which it has been dimensioned, has been to examine the following:

- The use of concrete bearing elements, arranged on the side with the higher displacements;

- Construction loading on the side opposed to the one with the highest settlement;
- Construction discharge on the side with high settlements.

The first measure for stopping the development of uneven settlements was to create a vertical screen, on the foundation interior outline, in the rooms where the soil repression presented a visible active evolution, in the area with the highest settlement speeds. The application of this measure has pursued maintaining the building support conditions on the soil and avoiding the manipulation of important deadweight masses or the demolition of certain building parts.

Considering the existing conditions (the space in the basement with limited height, the obligation not to discharge the repressed soil from basement) has been decided to create a screen. The screen has been realized from coupons of metallic pipes F 3?, from compartments of about 1,00 m, assembled with metallic plugs and introduced in soil by beating. The working place has been created by digging the repressed soil over the footing bush (Figure 5).

Figure 5. Creating the working place and introduction in soil of the pipes that constitute the stopping screens

During the execution, the engineer has created equipment for thrusting the pipes, by pressure, which determined a substantial increase in the installation speed of the stopping screens.

The soil repression stopping screens, situated in the basement, had a depth of 3,00-4,00 m from the foundation superior part.

By applying the stopping measures for the process of uneven progressive settlement, the settlement speed has strongly decreased, without cancelling the general rolling tendency of the construction on the street and laterally, towards the CL 16 building.

6. MEASURES FOR REESTABLISHMENT OF BUILDING VERTICALITY

By uneven settlement, the construction had reached at a settlement difference of about 65 cm corresponding to a leaning angle reported to the vertical of about 2°.

Considering the general aspect of the construction and its functionality (firstly the elevator functioning conditions) was necessary to find solutions for the reestablishment of the verticality.

Have been examined possibilities to straighten the building by lifting it, but the solution has been abandoned due to the high weight, the risks determined by the local rest and the lack of a rigid and resistant support in order to locate the hydraulic press for the lifting.

Finally, has been used the procedure of settlement straightening, by excavating near and under the construction footing, till determination of controlled processes of local repressing, which might have leaded towards general settlement standardization.

The diggings near the foundations have been realized at the interior, the repressed material volumes being maintained untouched in order to realize a section more rigid of leaning around which the footing to rotate, reducing this way to minimum the settlement of the building side towards the street.

At the exterior, the marginal excavations around the footings, till their base, have allowed the creation of local repressing which contributed to the building progressive straightening.

The building controlled displacement, towards straightening, has been determined by:

- Topographic measures, on the landmarks fixed in the building base;
- A suspended pendulum, to the superior part of the elevator shaft.

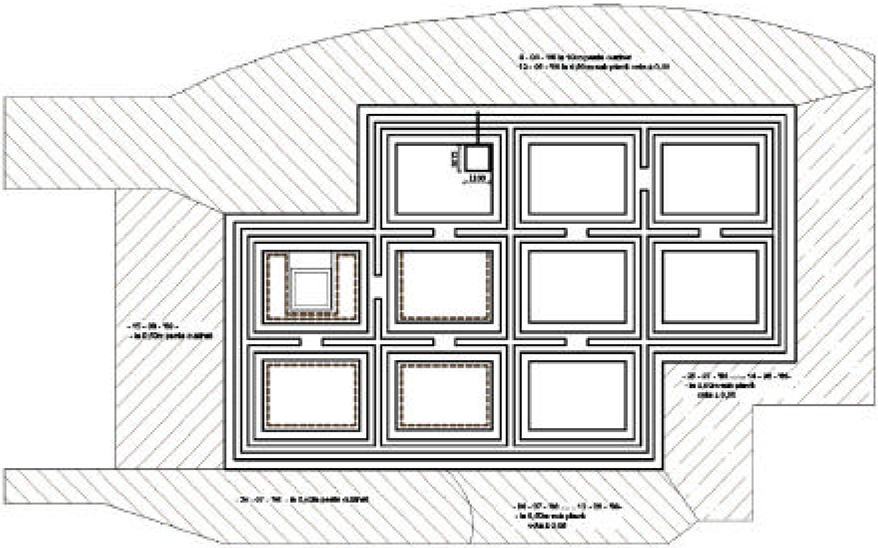

Figure 6. The digging plan executed at the interior and the exterior

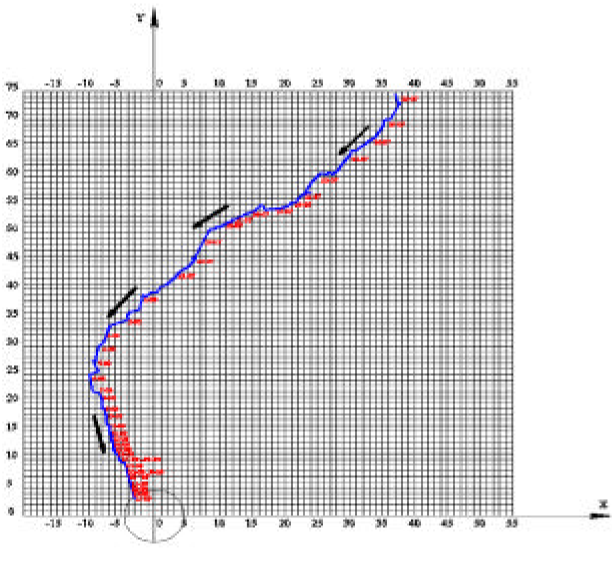

Figure 7. The pendulum direction from the elevator shaft

After straightening, the building has been equipped with a mat footing capable to receive the construction loading, without high uneven deformations and without soil repression.

For this operation has been acted this way:

- Before completing the straightening has been stopped any excavation. Has been realized all around the building line, the treatment with lime columns, cement lime and lime with cement and sand;
- The repressed soil has been evacuated on strips of approximately 0,60 m width;
- On each top soil remove at approximately 15 cm above the foundation plan has poured, on strips, a plate from concrete reinforced having an equalizing role for each room foundation plate. The mat foundation transmits to the structure, the soil reaction through marginal interior walls. These represent protection wall for the construction basement;
- By continuing the digging operations have been realized the footing straightening. Finally the reinforced mat foundation getting into work determined the reach of vertical position by elevator shaft axe.

7. CONCLUSIONS

All the observations established during the repressing phenomena have proved the building uneven progressive settlement.

The researches show that the clay on which was realized the building footing has characteristic properties which can be established by special geotechnical tests.

Has resulted a great influence of the humidity variation on resistance shear strength manifestation of Bahlui clay in different conditions of the soil stress state. Consequently to these phenomena the buildings realized on fat clay will have the footings adapted to the soil and structure characteristics. Is necessary the water remove from site and foundation soil quality improvement.

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