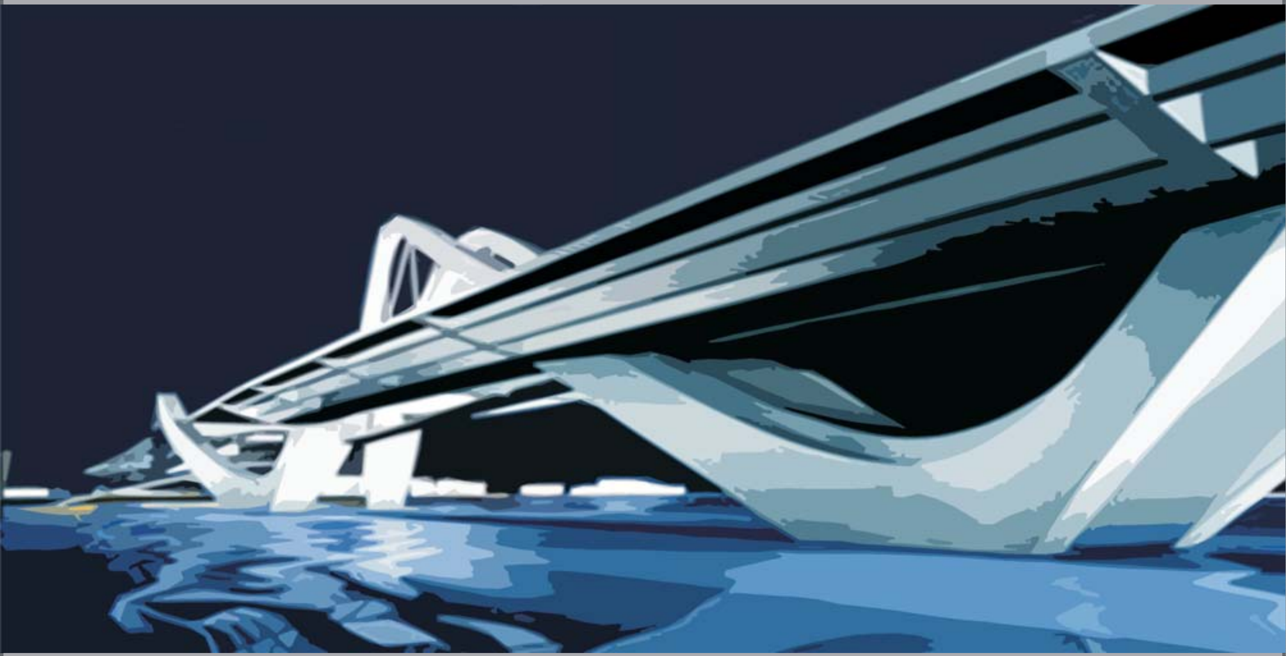
Transportation infrastructure new developments

C. Ionescu, R. Scînteie

editors



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TRANSPORTATION INFRASTRUCTURE NEW DEVELOPMENTS

Constantin Ionescu
Rodian Scînteie
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Design, Materials and Construction Analysis of a Concrete Pavement

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Summary

The paper presents the performance analysis of a concrete pavement, conclusions of the investigation and recommendations made for ensuring its durability. The main causes of imperfections were established by an investigation performed at the request of the contractor of a concrete pavement made at the platform of an industrial company. The platform covers around 200 000 m^2, and was built in 2002. Since than, numerous maintenance works were done, especially by replacing the damaged panels. The purpose of the investigation was to identify the causes that led to an unacceptable performance of the concrete pavement claimed by the owner, and to recommend measures in order to ensure the future normal service of the pavement.

KEYWORDS: concrete quality, structural and functional performance, maintenance

1. INTRODUCTION

Concrete pavements can be designed for virtually any service life, from as little as 10 years to 60 years or more. The primary factors in the design life are the materials quality and slab thickness. However, pavement concrete mixtures with enhanced strength and durability characteristics should be combined with a performant conception and structural design to ensure long service periods.

The platform situated in the industrial zone of a Transylvanian city covers around 200 000 m^2, and was built in 2002 (Figure 1). Since than, numerous maintenance works were done, especially by replacing the damaged panels.

Concrete is able to provide a highly durable, serviceable and attractive surface, but the quality of concrete pavements is often affected by conditions over which the designer and contractor have little control.

By keeping the causes of the imperfections in mind, it is possible to reduce the probability of unsatisfactory results.

Figure 1. The investigated concrete pavement

Contractors are not necessarily responsible for all imperfections. Poor design, unsatisfactory mixture proportions and improper service conditions are also implied. Some curling and cracking can be expected on every project due to the inherent characteristics of Portland cement concrete such as shrinkage.

2. DESIGN ANALYSIS

Various levels and frequencies of constructability reviews can be conducted, depending on the purpose and complexity of the project. Briefly, the problem is related to the whole life cycle analysis, so the decision has to be taken by choosing between the following two possibilities:

- Initial low cost pavement with important maintenance costs;

- Expensive initial investment with low maintenance costs.

The main objective of pavement design is to select pavement primary features, such as slab thickness, joint dimensions and reinforcing system to ensure the load transfer requirements, which will economically meet the needs and conditions of a specific paving project.

The goal of all pavement design methods is to provide a pavement that performs well. That means, to provide a serviceable pavement over the design period for the given traffic and environmental loadings.

A pavement desired performance is generally described in terms of structural performance and functional performance:

- Structural performance is the ability of the pavement to support current and future traffic loadings and to withstand environmental influences.

The structural performance of concrete pavements is influenced by many factors, including design, materials, and construction. The most influential design-related variables for structural performance at a given level of traffic are slab thickness, reinforcement, concrete strength and support conditions. The most prevalent type of structural distress is load-related cracking, which may appear as corner cracks, transverse cracks, or longitudinal cracks.

- Functional performance refers to the pavement ability to provide users a comfortable ride for a specified range of speed.

Most often, functional performance is thought to consist of ride quality and surface friction, although other factors such as noise and geometrics may also come into play. Functional distress is generally represented by a degradation of a pavement driving surface that reduces ride quality.

Both structural and functional distresses are considered in assessing overall pavement performance or condition. Even well-designed and well-constructed pavements tend to degrade at an expected rate of deterioration as a function of the imposed loads and/or time. Poorly designed pavements (even if they are well-constructed) will likely experience accelerated deterioration. Regarding to the above presented aspects, the analysis of the investigated pavement will be made as following.

There are three basic types of pavement construction. Each of these design types can provide long-lasting pavements that meet or exceed specific project requirements.

- Jointed plain concrete pavements: Because of their cost-effectiveness and reliability, the vast majority of concrete pavements constructed today are of this type. They do not contain reinforcement, and have the transverse joints generally spaced less than 6.5 m apart. They may contain dowel bars across the transverse joints to transfer traffic loads across slabs and also may contain tie bars across longitudinal joints to promote aggregate interlock between slabs.

- Jointed reinforced concrete pavements: This type of pavement contains both joints and reinforcement (e.g., welded wire fabric, deformed steel bars). The joint spacings are longer (typically about 9 to 12 m), and the dowel bars and tie bars are used at all transverse and longitudinal joints, respectively.

The reinforcement, distributed throughout the slab, composes about 0.15 to 0.25 percent of the cross-sectional area and is designed to hold tightly together any transverse cracks that could develop in the slab.

It is difficult to ensure that the joints are cut where the reinforcement has been discontinued.

- Continuously reinforced concrete pavements: They do not have any transverse joints, but they contain a significant amount of longitudinal reinforcement, typically 0.6 to 0.8 percent of the cross-sectional area. Transverse reinforcement is also often used. The high content of reinforcement influences the development of transverse cracks within an acceptable spacing (about 0.9 to 2.5 m apart) and serves to hold cracks tightly together. Some agencies use CRCP designs for high-traffic urban routes, because of their suitability for high-traffic loads.

The constructive system of the investigated pavement is an extension of the jointed plain concrete system (Figure 2), a bottom reinforcing mesh being introduced in order to sustain tensile stresses induced by heavy traffic. No information about the analytical design and durability design were received. A cross-section of the pavement under investigation, declared by the contractor and verified on site, is shown in Figure 3. Joints are placed correctly, delimiting panels with 1:1 aspect ratio (4.0×4.0 m and 5.0×5.0 m). Saw cut joints are 7 cm deep.

The pavement is framed by EN 206 provisions in the exposure class XF3 in relation with freeze-thaw attack (description of environment: high water saturation, without de-icing agents). Thus, related to the performance of the pavement, following remarks are necessary:

- The very small slope of 0.3% (e.g., recommended slopes are 2-3%, but not less than 1%), which leads to a very poor surface drainage. Pavements are exposed to severe humid environment conditions, and poor drainage amplifies the severity of the service conditions, accelerating aging of concrete and degradation of the pavement system;

- For the exposure class XF3, EN 1992 recommends the concrete class C 30/37 to ensure the proper durability. In the design, the concrete class C 16/20 was adopted, which presume less shrinkage but a lower permeability and more sensitivity to freeze-thaw cycles. According to the contractor, the choice of a lower class was taken in relation with the constructive solution, considering as target minimum shrinkage (and consequent lack of specific reinforcement – see paragraph below) since the strength is sufficient;

- The lack of top reinforcement suggests that the solution was adopted for a low initial investment. Concrete can not sustain tensile stresses and cracking of concrete pavement is inevitable. Thus, the system has no reinforcement which should keep within the acceptable limit (less than 0.2 mm) the crack widths, and visual inspection emphasized that this represents the major distress. Unsymmetrical disposed reinforcement also provides a higher restraint at the bottom of the slabs, giving support through the induced parasite tensile stresses to the occurrence and development of the cracks.

The consequence of this conception is that the pavement needs more often repair works by injecting and sealing the cracks in order to maintain its performance;

- The lack of dowels presumes a regular stiffness of the subbase and subgrade, with low softening sensitivity. The subbase level is below the minimum freezing depth (-0.80 m) in that area. Nevertheless, visual inspections revealed that practically there are no problems with the subbase and subgrade softening and frost heave. The satisfactory compacting of the subbase and subgrade is confirmed.

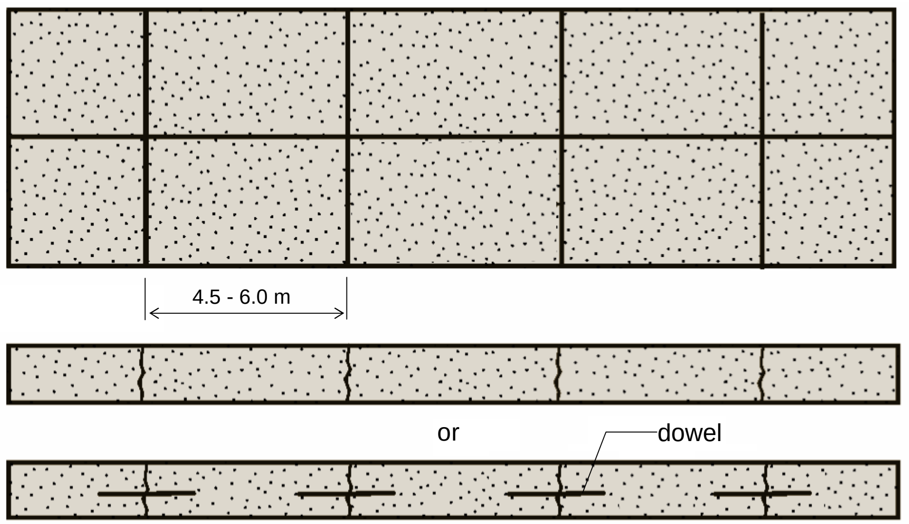

Figure 2. Jointed plain concrete pavement

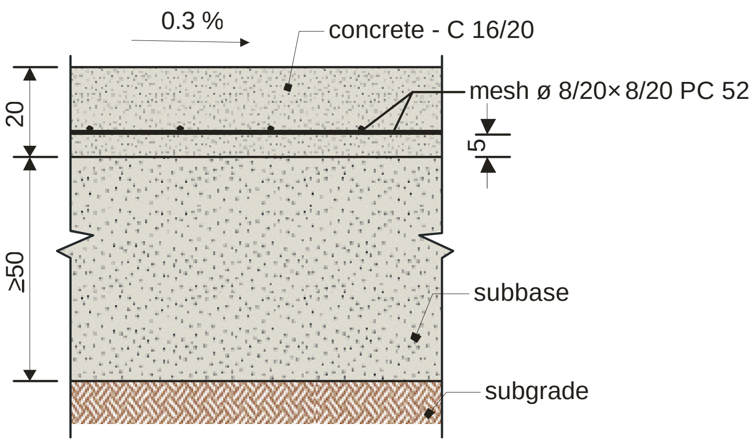

Figure 3. Detail of the pavement

3. MATERIALS AND CONSTRUCTION ANALYSIS

If compressive strength is enough to sustain the mechanical actions, and the structural performance was ensured at the time of the investigation, problems refer to the rest of the properties that influence the global performance of the pavement.

Mix design is fundamental in order to ensure a good quality concrete. The contractor did not specify any reference mix, mentioning that there were several suppliers for the fresh concrete. Visual inspections revealed that the most important factor of distress for the functional performance of the pavement is excessive cracking.

Studies revealed the short and long term evolution of the shrinkage strain for a typical C 16/20 mix, with 350 kg/m^3 of cement, and various weather conditions at concrete placing, both for Portland cement I 32.5 R and composite Portland cement II AS 32.5 R (with slag 6-20 %).

Peak temperatures are reached at 12-16 hours age of concrete, and the maximum temperature change and associated axial strains caused by volume contraction are shown in Table1.

Table 1. Reference temperature changes and volume contraction at the C 16/20 pavement

Weather conditions	Cement type	Maximum effective temperature change [ºC]	e_v [‰]
cold	I 32.5 R	12.5	-0.125
moderate	I 32.5 R	9.6	-0.096
hot	I 32.5 R	11.2	-0.112
cold	II A-S 32.5 R	10.0	-0.100
moderate	II A-S 32.5 R	7.7	-0.077
hot	II A-S 32.5 R	9.0	-0.090

The inevitable cracks occur within the first week from concrete placing. Analysis crack widths reach about 0.4 mm in the case of cement I 32.5 R, and about 0.26 mm in the case of cement II A-S 32.5 R, both more than the allowable limit 0.2 mm. Considering the results of the analysis and the observations of visual inspections, appears obvious that the concrete mix contains cement type I 32.5 R and has a high w/c ratio, with greater shrinkage potential.

During repair works, the contractor replaced plain concrete with steel fibers reinforced concrete (20 kg/m^3), in order to reduce cracking. Because steel fibers are dispersed on the entire volume of the slab, this quantity is not enough to reduce significant the crack widths, its effect being a global increase of the ductility of concrete, property that practically has no significance for the analyzed problem.

Tests on core samples reveal that in some areas, concrete compressive strength is beyond the one prescribed by the design.

It should be noticed that due to the uncertain age of the tested concrete, this problem could be justified also by accelerated aging of concrete. However, supplementary analyses are necessary in order to determine if some of the cracks that are currently justified by early or heavy traffic, are caused practically to the insufficient strength of concrete.

Concerning the construction, it should be mentioned that visual inspections revealed a careful made construction process. No major imperfections can be justified by poor construction practice, even if it was performed and maintained in various weather conditions.

Another important distress factor is scaling and delaminating of concrete. This is obviously caused by the sensitivity of the superficial layer to freeze-thaw cycles, consequence of a high w/c ratio and great permeability (should be mentioned that the small slope is not correlated with the necessary properties).

Due to the severe exposure conditions, besides mechanical strength, other properties of the hardened concrete are critical and have to be considered:

- Permeability: concrete with permeability classes P8 (200 mm water penetration at 28 days under a pressure of 8 bars) or P 12 (300 mm water penetration at 28 days under a pressure of 12 bars) is commonly used in pavements;

- Gelivity (freeze-thaw resistance): concrete with freeze-thaw resistance from 100 cycles, up to 200 cycles are preferred for long term durability.

4. CONCLUSIONS AND RECOMMENDATIONS

The investigation made upon the concrete pavement is pointing out following conclusions:

At the time of investigation, the structural performance of the pavement was good;

Functional performance presents two important distress factors:

- Excessive cracking, due to restrained shrinkage of concrete and early and/or heavy traffic;

- Low freeze-thaw resistance, causing scalling and delamination of the superficial layers;

The degradation mechanisms related to the above mentioned distress factors are:

- Excessive cracking of concrete allows easily water to penetrate inside concrete. Low temperatures during winter transform the water in ice, which enlarge the initial volume and exerts pressure upon the surrounding concrete.

Thus, microcracks are developed and in short periods of time initial cracks present much larger openings. This aspect affects the functional performance, and in time also reduces the structural performance beyond the acceptable limit.

- High permeability allows water to penetrate the concrete, resulting in scalling and occurrence of new cracks that grow further following the above mechanism; high w/c ratio also led to significant delamination;

In order to ensure the future performance of the pavement and a reasonable durability, the recommendations are:

- Injection of the cracks with widths more than 0.2 mm at least once per year;

- Replacement of the panels with affected areas more than 80 % with a concrete based on a new mix design. Recommended concrete classes are C 25/30 or C 30/37. Recommended cement is II AS 32.5 R, and maximum water content derived from w/c = 0.42. Air entraining admixtures to reduce porosity are also recommended;

- Because the shrinkage potential remains high, instead of injecting and sealing the excessive cracks, reinforcing of the superficial layer (at least 0.2 % reinforcing ratio) with a two-directional mesh is recommended.

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Steel fibre reinforced roller compacted concrete Roads

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Summary

This paper describes the use of steel fibres in Roller Compacted Concrete (RCC) for pavement applications. This work is part of the EU funded project Ecolanes, which aims to develop long lasting rigid pavements. It begins by discussing the benefits associated with rigid pavements with a special reference to fibre reinforced concrete and RCC. The objective of the specific part of the presented work is to investigate the effect of adding fibres in RCC from a structural standpoint, and to discuss the improvement in strength and toughness characteristics, and comment on the effect that different fibre types may have as reinforcement. The importance of fibre geometry on the flexural behaviour of steel fibre reinforced RCC (SFR-RCC) is demonstrated. RCC reinforced with fibres obtained from industrial sources and fibres obtained from the bead wire of recycled tyres, resulted in better flexural behaviour, compared to RCC reinforced with tyre cord fibres recovered from recycled tyres, at equivalent fibre ratios. However, the recycled tyre cord fibres could be a viable alternative if used at higher fibre volumes.

Keywords: Pavement, Roller Compacted concrete, Fibre reinforced concrete, Recycled materials.

1 INTRODUCTION

A significant and targeted investment (approximately €600bn by 2010) is currently required for the rehabilitation and extension of the European surface transport infrastructure, to provide an adequate system to respond to the needs of the enlarged European Union, for the benefit of the single market and economic and socio-economic integration [1,2]. The core element of surface transport infrastructure is the pavement, which can be either flexible or rigid. Flexible pavements are normally constructed with asphalt, whereas Portland Cement Concrete (PCC) is the main material used in rigid pavements, which may be reinforced either with conventional steel mesh or steel fibres.

With rapidly escalating oil prices the future of flexible pavements, which require deep foundations and deep asphalt layers, is becoming progressively more uncertain due to increasing costs as well as political and environmental concerns. Although conventional concrete pavements can significantly reduce the foundation layers, decrease or eliminate the asphalt topping and reduce maintenance requirements, they are currently associated with higher costs and increased construction complexity due to the use of reinforcing mesh. Steel fibre reinforced concrete (SFRC) could eliminate the use of mesh reinforcement and allow the use of the roller compacted method to lay the concrete, bringing significant savings in both time and money in pavement construction. Nevertheless, in order to provide an economical and sustainable solution for concrete pavements, it is necessary to utilise materials coming from various waste streams and to carefully evaluate their energy requirements as well as the cost of maintenance.

In the next sections follows a discussion on the benefits associated with concrete pavements with special reference to steel fibre reinforced concrete (SFRC) and RCC. After that follows a detailed description of the bending tests performed, including details of the different types of fibre reinforcement involved. Finally a discussion on the main conclusions drawn from the bending test results is presented.

2 RIGID, FLEXIBLE OR A HYBRID PAVEMENT?

Rigid pavements have a longer working life (approximately 40 years) outlasting flexible materials by approximately 20 years. In addition rigid pavements require less maintenance, whilst when repairs are necessary they are typically smaller in scope than for flexible pavements [3]. Furthermore, in presence of heavy vehicles, flexible pavements experience greater deflections compared to rigid pavements, causing 11% higher vehicle fuel consumption [4]. Additionally, the effect of seasonal changes in the smoothness of rigid pavements is significantly lower, which also contributes to higher fuel efficiency. Rigid pavements due to their stiffness can withstand even the heaviest traffic loads without suffering distress (e.g. rutting and shoving) common with flexible pavements. Rigid pavements continuously gain strength over time, and quite often exceed their design life expectancy as well as the design traffic loads. Moreover, restoration techniques can extend the life of rigid pavements up to nine times their original design life [3,5].

A key feature of rigid pavements is that they can be reinforced and minimise cracks which, in combination with their high material stiffness, reduces the required pavement thickness to carry the expected traffic loads, and in many cases a solution is achieved with single layer construction.

In terms of cost, the price of PCC has remained relatively stable over recent years, while hot mix asphalt prices have risen significantly. This trend is anticipated to continue. In particular, the Producer Price Index (PPI) for liquid asphalt at the refinery (as tracked by the U.S. Bureau of Labour Statistics) had risen by February 2007 nearly 37% over the preceding 12 months, compared to nearly 6% for cement. The PPI for asphalt paving mixes had increased just over 27% in the 12 months ending December 2006, while ready mixed concrete had risen nearly 10% [6].

The main limitations associated with rigid pavements are poor surface skid resistance, noise generated from traffic and their relatively high current cost [7]. However, a combination of a rigid pavement with an ultra thin asphalt overlay can enhance the ride quality of the pavement, without the problems traditionally associated with deep asphalt pavements, and this is an attractive option. Despite the increasing prices of asphalt, the material cost of rigid pavements may still be higher than that of asphalt pavements and, hence, at this moment in time an economical and sustainable solution for rigid pavements, could be derived through the utilisation of low-cost materials and by making use of processes (eg. roller compaction) that are fast and economical.

2.1. Roller compacted concrete

Figure 1, depicts a typical construction scene of pavement construction that is used in nearly any street of every city. A careful inspection of Figure 1, however, confirms that the material in use is grey concrete, not asphalt, but it is not conventional concrete either. It is a material called Roller Compacted Concrete (RCC), named after the technique used to compact it.

RCC is a construction material made from the combination of aggregates, water and binder. Although the same ingredients as for conventional concrete are used, these are mixed at different proportions resulting in a material with improved properties and behaviour. The ingredients are mixed in a central batching plant or in a Pugmill mobile mixer, targeting the production of a heterogeneous mass which has consistency representative of zero slump concrete [8]. This is needed so that the concrete can support the weight of vibratory rollers (approx. 10tons) as soon as it is layed in a similar way to asphalt. It can be said that asphalt is a form of concrete with bituminous materials replacing cements.

Figure 1: Paving operation and compaction of the pavement by heavy vibratory rollers

RCC aims to provide the relatively high strength and durability of concrete, at the economy and speed of construction traditionally associated with asphalt. This very attractive material could further benefit from utilisation of fibre reinforcement.

2.2. Steel fibre reinforced concrete

Fibres are added in concrete to minimise cracks, to enhance toughness and improve surface characteristics. In particular, the addition of steel fibres can offer high first crack flexural strength, improved shear strength, high flexural fatigue endurance limit, good impact resistance, enhanced freeze-thaw durability, ability to carry load after the formation of cracks and high spall resistance. SFRC can also benefit from thickness reduction without experiencing the problems of curling and warping as may be seen in high cement content, thin slabs [9].

From the construction perspective, the use of steel fibres, can eliminate the need for conventional re-bars and, hence, lead to cost savings and increased speed in construction [10,11].

3. BENDING BEHAVIOUR OF STEEL FIBRE REINFORCED RCC

Bending tests were performed on rectangular prisms to evaluate the flexural strength characteristics (toughness) of SFR-RCC. The next sub-sections present a description of the steel fibre reinforcement investigated, the specimen preparation technique used, the testing procedure followed and the results obtained from the bending tests performed on rectangular SFR-RCC prisms.

3.1. Steel fibres extracted from post consumer tyres

Steel tyre-cord fibres, produced from the mechanical treatment (e.g. shredding and granulation) of post-consumer tyres, have high variability in length and diameter and in most of the cases they contain significant amounts of rubber particles on their surface.

One of the main problems, encountered when mixing steel tyre-cord fibres in fresh concrete, is the tendency of the fibres to ball together, which spoils the concrete [12]. To avoid balling and to optimise the use of such fibres in concrete, the fibres need further treatment to remove the rubber particles and minimise the geometrical irregularities [13].

Three types of recycled tyre cord fibres, with different length distributions, were considered in this investigation. RTC1-11 fibre type had a length range between 1-11mm, while RTC1-17 ranged between 1-17mm and RTC3-40 between 3-40mm. The RTC fibre types had an average diameter of 0.23mm and a tensile strength of around 2000MPa. In addition to the RTC fibres a fibre made from the tyre bead wire used, namely I2H2x1/80. The I2H1x2/80 is a fibre with hooked ends, a rectangular cross section of 1x2mm and an average length of 80mm with a tensile strength of around 2000MPa.

3.2. Industrially produced steel fibres

The four different types of industrially produced steel fibres used in this study were I2H1/50, I2H.75/50, I2H1/60 and I2C1/54. I2H, is a loose cold drawn wire fibre with hooked ends, Figure 2a. These fibres are filaments of wire, deformed and cut to lengths, for reinforcement of concrete. The geometrical properties of the fibre are 50mm and 60mm in length, and 1mm in diameter, with a tensile strength of around 1100MPa. I2C1/54 fibres have the same properties as I2H type fibres with the only difference being their cone-ends, Figure 2b.

(a) (b)

Figure 2: Industrially produced fibres: (a) I2H; (b) I2C1/54

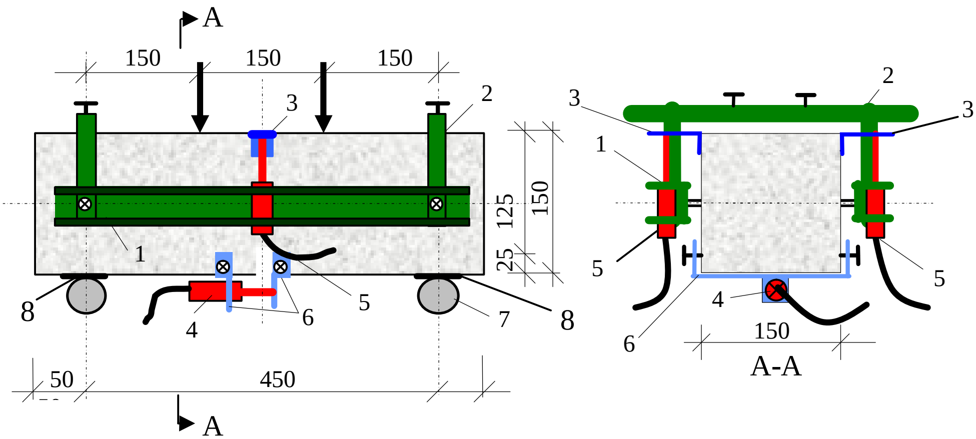
3.3. SFRRCC specimen preparation

The SFRRCC prisms were 150mm deep, 150mm wide and 550mm long. Steel-plate moulds were used to eliminate the deformation of the moulds, caused by the severe external compaction. The specimens were cast in three layers and compacted by a suitable vibratory kango hammer.

Following the recommendations of the RILEM bending test [16], a day after casting, the specimens were demoulded and then placed in the mist room ($+20°C$ and $RH \geq 95$) until the day of testing. On the day of testing, a notch (25mm high and 5mm thick) was sawn at mid-span, into the tensile face of each rectangular prism (at a 90 degrees angle to the RCC layers), using rotating diamond blades. The purpose of the notch was to act as a crack inducer [13].

3.4. Testing procedure

Testing of the notched prisms was carried out by following the recommendation of the RILEM bending test [14]. It is noted that a four-point load arrangement was used instead of three-point load. The use of four-point load arrangement creates a region of constant moment and, hence, minimises the overestimation of bending resistance, caused at the point of load application by the load-spreading effect [15]. The two supports and the device for imposing deformation consists of steel rollers with a diameter of 30mm. Two rollers (one at the support and one at the device imposing the deformation) are capable of rotating freely around their axis and the longitudinal axis of the test specimen [14]. All rollers are placed on steel plates (5mm thick) to avoid local crushing of concrete and extraneous deformations.



1- Steel bar; 2- clamps with pins; 3- steel plate (glued to the prism); 4 - LVDT4; 5 – LVDT5; 6- clamps for LVDT; 7- supports, 8 - steel plates
Figure 3: Set-up used for the bending test for RCC prisms

Results from bending tests on concrete prisms are prone to significant experimental errors (due to spurious support displacements, machine stiffness and load rate) and, hence, extra care is required to obtain accurate deflection measurements [16]. To avoid these errors and the effect of torsion on the deflection measurements, a yoke was used as specified by the Japan Society of Civil Engineers [17]. Average midspan beam deflections were measured on both sides of the prisms using two transducers fixed to the yoke (LVDT5) and, hence, any torsional effects were cancelled out. One transducer (LVDT4) was mounted across the notch mouth to monitor the crack mouth opening displacement (CMOD), as illustrated in Figure 3.

The SFRRCC specimens were tested in a 100 kN servo-hydraulic machine under crack-mouth-opening-displacement control (CMOD). The machine was operated in such a manner that the CMOD was increased at a constant rate of $60\,mm/\min$ for CMOD ranging from $0-0.1mm$ and $0.2\,mm/\min$ for CMOD from $0.1mm$ until the end of the test [14].

3.5. BENDING TEST RESULTS

Overall RCC reinforced with any type of steel fibres presented improved performance compared to plain RCC, with some fibres being more effective than others depending on their specific characteristics. The effect of adding fibres on the flexural behaviour of RCC is illustrated in Figures 4, 5 & 6. Figure 4 shows the flexural behaviour of RCC specimens reinforced with four different types of industrial steel fibres; while, Figure 5 shows the flexural behaviour of specimens reinforced with steel tyre-cord and hooked-end bead wire fibres. In Figure 6, the bending behaviour of RCC with high fibre volumes of RTC fibres is presented.

At equivalent fibre ratios specimens with relatively long steel fibres (i.e. length > 50mm) exhibited an extended and more stable post-peak load-vertical displacement response, contrary to the limited vertical displacement obtained by the specimens with 1-11mm and 3-40mm long fibres. A comparison between the two short recycled fibre types (RTC1-11 and RTC3-40) further reinforces this observation as the longer RTC3-40 fibres show an improved post-cracking behaviour.

The effect of fibre shape on the toughness can also be observed in Figures 4, 5 & 6. Specimens reinforced with fibres having deformed ends presented higher modulus of rupture and better post-cracking behaviour at equivalent fibre ratios. This is largely attributed to the beneficial effect that the deformed ends have on mechanical bond, dramatically increasing SFRRCC ductility.

Flexural failure of the specimens occurred mainly due to fibre pullout. It is noted that, in specimens reinforced with the I2C1/54 fibres, up to 50% of the fibres experienced fracture at their ends prior to pull out; increasing the energy absorption capacity of their specimens. I2C1/54 fibre fracture is an indication of the high involvement of this type of fibres, demonstrated by hardening behaviour, at fibre ratios higher than 2% by mass ($50kg/m^3$).

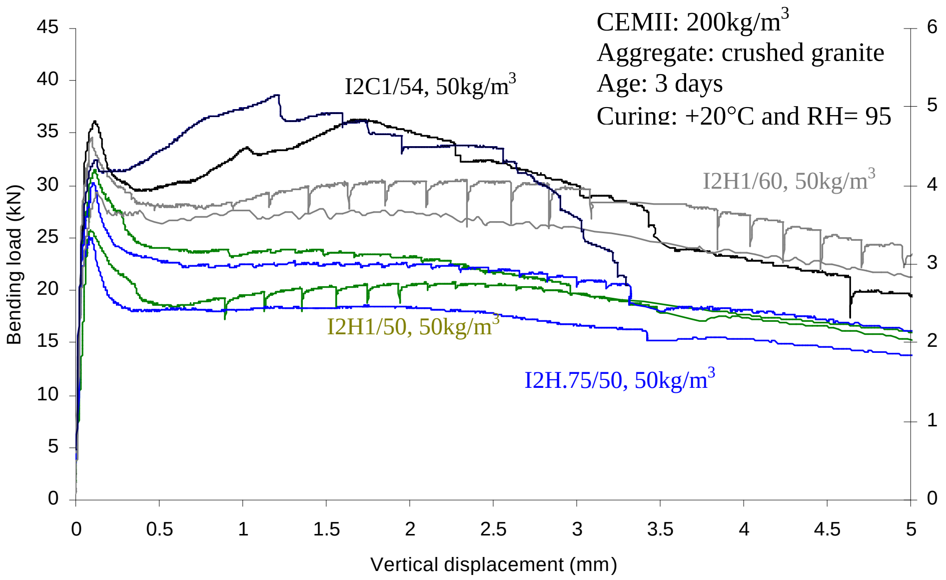


Figure 4: Flexural behaviour of SFR-RCC utilising industrial fibres (3-day test)

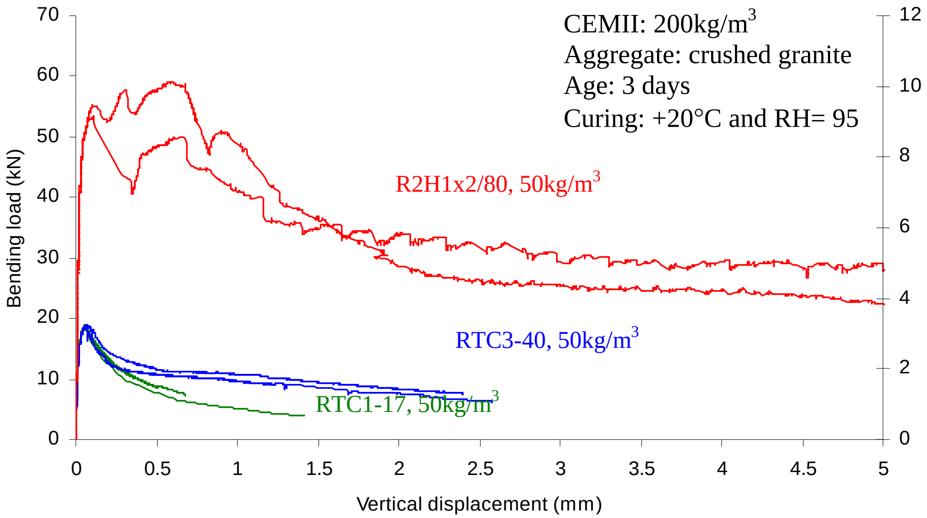


Figure 5: Flexural behaviour of SFR-RCC utilising recycled fibres (3-day test)

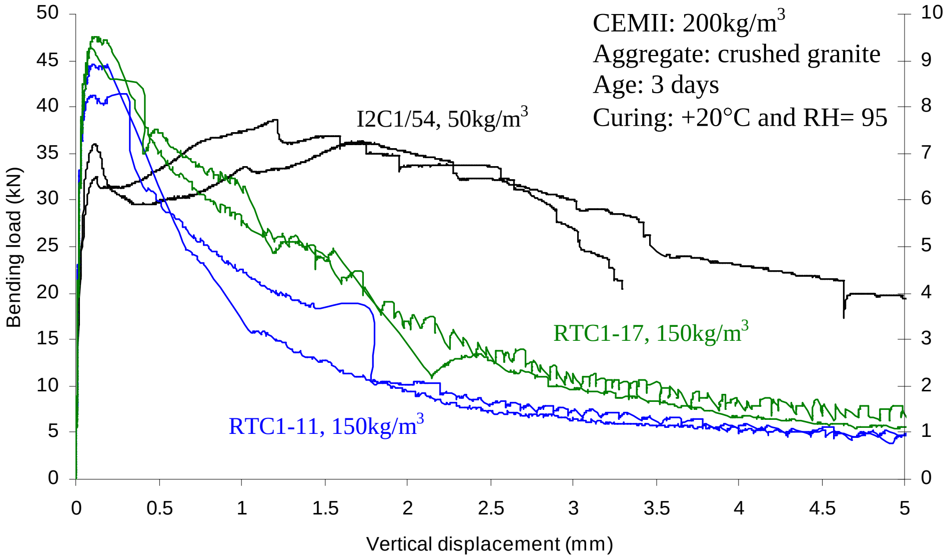


Figure 6: Flexural behaviour of SFRRCC utilising high fibre volumes of RTC fibres (3-day test)

From Figure 6, it is apparent that by increasing the RTC fibre content by approximately three times the flexural strength could be doubled and the residual strength following the initiation of cracking, significantly improved. The flexural peak strength achieved by specimens reinforced with RTC fibres at fibre contents of 150kg/m^3 were superior to the flexural strengths achieved with 50kg/m^3 of industrial fibres and comparable to the strengths achieved with R2H1x2/80 fibres. The efficiency issue with the relatively short RTC fibres could be compensated by their low price. The price of industrially produced steel fibres is ranging from €700 to €15,000 per tonne (at least 20% higher than the price of conventional steel bars), while the RTC price is between €50 to €150 per tonne.

4. CONCLUSIONS & FUTURE WORK

The increasing demand to adopt innovative, sustainable as well as cost-effective construction practices leads to the wider use of concrete pavements. RCC offers the high strength and durability of concrete, at the economy and speed of construction traditionally associated with asphalt. The utilisation of fibre reinforcement brings significant benefits in construction processes and improves significantly ductility characteristics and the modulus of rupture of RCC which is the main property of interest currently in pavement design.

The importance of fibre geometry on the flexural behaviour of RCC has been demonstrated, with length and shape having significant effect. The fibres examined from industrial sources and the recycled bead wire fibres resulted in better overall behaviour compared to the short RTC fibres, at equivalent fibre ratios.

Although, the short tyre cord fibres presented an inferior behaviour compared to the industrial fibres, if they are mixed at higher fibre ratios (approximately 3 times higher) they could be used as a viable alternative to the expensive industrial fibres.

Further work needs to be concentrated on examining several different RTC fibre length distributions to assess their performance in RCC. This is a necessary step on the implementation of these fibres as different tyre shredding plants use different tyre shredding processes resulting to variable fibre length distributions.

5. ACKNOWLEDGMENTS

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The Consolidation of Steel Bridges Superstructures Without Introducing Initial Stress States

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Summary

Steel bridges are structures with an extended lifetime and long-term performance but may be overloaded, as regards the bearing capacity, as a result of the working load increase over time. When the steel bridge floor does not have major defects or damages, between the solution of replacing it and the solution of consolidating it, the latter is preferred. It is estimated that the consolidation of a steel bridge floor could be considered effective when the consolidation cost does not exceed 30 -35% from the value of a new bridge floor.

The present work includes some consolidation solutions for the main girders of steel bridges. After comparing the results it can be established the most advantageous consolidation solution from a technical and economical point of view.

The following consolidation solutions have been taken into consideration:

The consolidation by enhancing the bottom flange of the girder section with chord plates applied directly on the bottomflange of the girder without cancelling out the permanent loads stress;

The consolidation by enhancing the bottom flange of the girder section with chord plates applied directly on the bottom flange of the girder by cancelling out the permanent loads stress;

The consolidation with unprestressed rigid steel tension rod applied under the bottom flange of the main girders.

KEYWORDS: steel bridge floor, consolidation, consolidation chord plates, unprestressed rigid steel tension rod.

1. INTRODUCTION

It is known that the steel superstructures of bridges have a long lasting operating time by comparison with concrete superstructures (especially those from pre-stressed concrete); they can easily exceed 100 years.

The maintenance of a steel superstructure during the operating time must be carried out accordingly (mainly the painting of the superstructure according to the maintenance schedule), so that the superstructure will not be affected by the damages. The difference of traffic loads between the initial design values and the real value at a given moment can lead to the exceeding of the bearing capacity. As a result a series of consolidation works are required in order to ensure the further use of the superstructure in safe conditions.

For instance, the superstructure of railroad bridges over the Danube at Felesti and Cernavoda, which were designed in 1889, was calculated for a convoy with a 13,00t per axle for locomotives and a distributed load of 4,5t per metre for carriages.

In the 1960s, after almost 65 years of operation, as a result of the increase of the traffic loads (an increase by almost 100%) in some stay rods of the Cernavoda bridge, were developed normal unit stress exceeding yield stress. Consequently the consolidation of the bridge floor was begun and the consolidation works were carried out as following:

- the enhancement of the bottom flange of the main girders by introducing a third unprestressed web plate;
- the enhancement of the diagonal bars section by adding pre-stressed or unpre-stressed fabricated elements;
- the installation of new longitudinal girders;
- the consolidation of the cross-bars with an unpre-stressed “railway switchgear” system;
- the consolidation of the top flange of the two main girders by introducing a third pre-stressed steel plate;

Below are presented three consolidation solutions for the main simple web girders of a bridge superstructure with an exceeded bearing capacity and a case study in which the methods used are being explained.

The three consolidation methods have in common the following:

- the consolidation involves the enhancement of the bottom flange of the girder, the access to the top flange is not possible in the case of a top-road bridge due to the bridge floor;
- adding new elements is done without introducing initial stress in the structure (pre-flexion, pre-tension, etc.);
- the consolidated bridge floor is assembled through riveting and the new introduced elements are attached also through riveting;

2. CONSOLIDATION METHODS

2.1. The consolidation by enhancing the bottom flange of the girder section with chord plates applied directly on the bottom flange of the girder without cancelling out the permanent loads stress

From a technological point of view the working stages are as follows:

- the rivet heads from the bottom flange of the girder are cut (without taking out the cut rivets) on the area on which new steel plates are to be attached;
- the new steel plates are placed in the correct position, regarding the position of the existing rivets;
- the cut rivets are taken out one by one and are replaced with the new ones, which are installed in the rectified holes;

As regards the calculation, it results the following stress states, which when combined give the final girder stress state:

a) On the unconsolidated section of the girders develops a stress state produced by the bending moment given by the weight of the unconsolidated structure and of the new introduced elements (Figure 1);

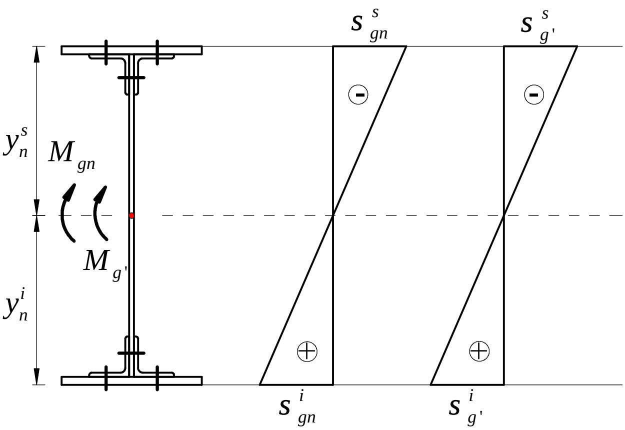

Figure 1

$$s^{s}_{gn} = \frac{M_{gn}}{I_{gn}} y^s_n ; s^{s}_{g'} = \frac{M_{g'}}{I_{gn}} y^s_n ; \tag{1}$$

$$s^{i}_{gn} = \frac{M_{gn}}{I_{gn}} y^i_n ; \quad s^{i}_{g'} = \frac{M_{g'}}{I_{gn}} y^i_n ;$$

b) The main girders are consolidated by enhancing the bottom flange section with steel plates. On the resulting section the bending moment given by traffic loads, produces a stress state illustrated in Figure 2.

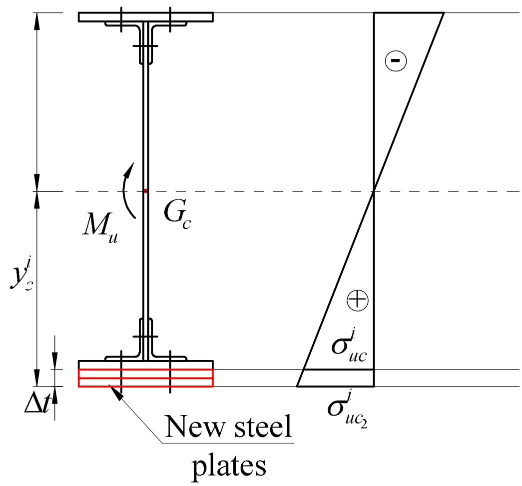

Figure 2

$$s_{uc}^{s} = \frac{M_u}{I_{gc}} y_c^s; \quad s_{uc_1}^{s} = \frac{M_u}{I_{gc}} (y_c^i - \Delta t); \quad s_{uc_2}^{i} = \frac{M_u}{I_{gc}} y_c^i; \qquad (2)$$

c) The final stress state on the consolidated section results when the two states presented are combined (Figure 3).

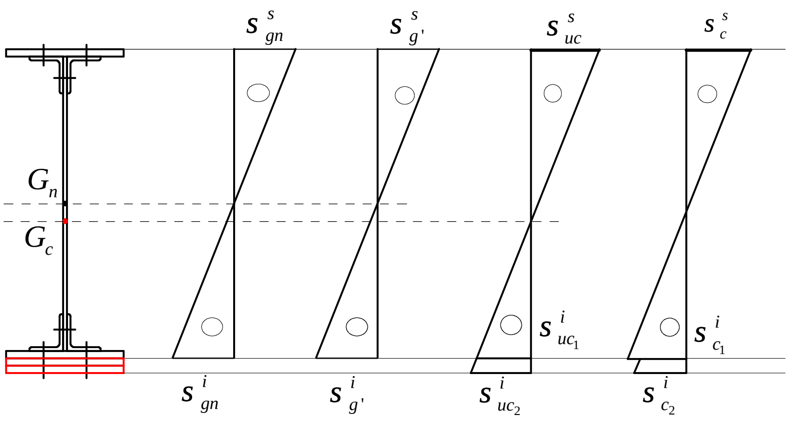

Figure 3

The strength condition for the consolidated structure is:

$$
\begin{aligned}
s_c^s &= s_{gn}^s + s_{g'}^s + s_{uc}^s \leq s_{an}; \\
s_{c_1}^i &= s_{gn}^i + s_{g'}^i + s_{uc_1}^i \leq s_{an}; \qquad (3) \\
s_{c_2}^i &= s_{uc_2}^i \leq s_{ac};
\end{aligned}
$$

2.2. The consolidation by enhancing the bottom flange of the girder section with chord plates applied directly on the bottom flange of the girder by cancelling out the permanent loads stress

The solution is applied if the permanent loads have a high value and consume an important part of the main girders bearing capacity.

The working stages are those mentioned at point 2.1., only that previously must be installed scaffolds under the main girders, thus any stress being eliminated. After adding new steel plates and disassembling the scaffolds, the consolidated section takes over all the loads – permanent loads and traffic loads.

The stress state is shown in Figure 4.

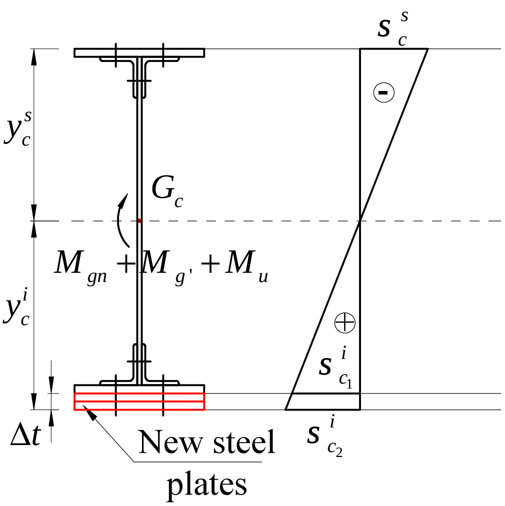

Figure 4

The strength condition for the consolidated structure is:

$$
s_c^s = \frac{M_{gn} + M_{g'} + M_u}{I_{gc}} y_c^s \leq s_{an};
$$

$$s^{i}_{c_1} = \frac{M_{gn} + M_{g'} + M_u}{I_{gc}} (y^i_c - \Delta t) \leq s_{an}; \tag{4}$$

$$s^{i}_{c_2} = \frac{M_{gn} + M_{g'} + M_u}{I_{gc}} y^i_c \leq s_{ac};$$

2.3. The consolidation with unprestressed rigid steel tension rod applied under the bottom flange of the main girders

This method involves the installation of a rigid steel tension rod under the bottom flange of the main girders, the rod is fixed at the ends of the girder. Inside the rigid steel tension rod applied is developed a tensile force, produced by the traffic loads (self-tensile force), which is determined on the girder-tension rod structure once statically indeterminate.

The stress states from the structure are the following:

a) On the unconsolidated section of the girders is developed a stress state produced by the bending moment given by permanent loads (Figure 5). The weight of the tension rod has not been taken into consideration;

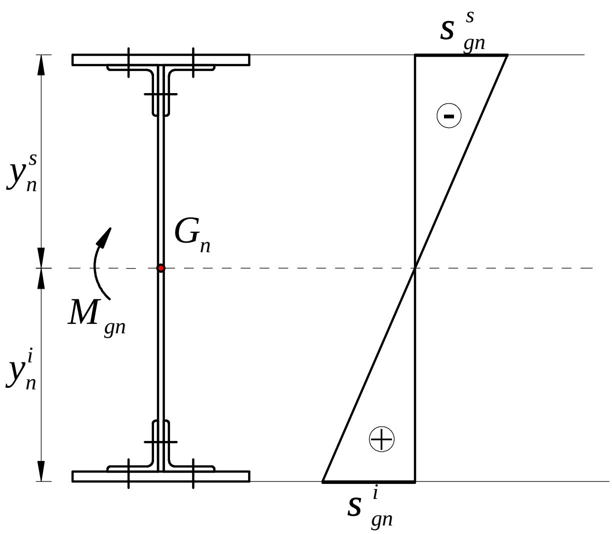

Figure 5

$$s^{s}_{gn} = \frac{M_{gn}}{I_{gn}} y^s_n; \; s^{i}_{gn} = \frac{M_{gn}}{I_{gn}} y^i_n; \tag{5}$$

b) On the consolidated section with the tension rod positioned at distance *e* towards the bottom flange of the girder is developed a stress state produced by the bending moment given by traffic loads and the tensile force (self-tensile force) from the tension rod (Figure 6);

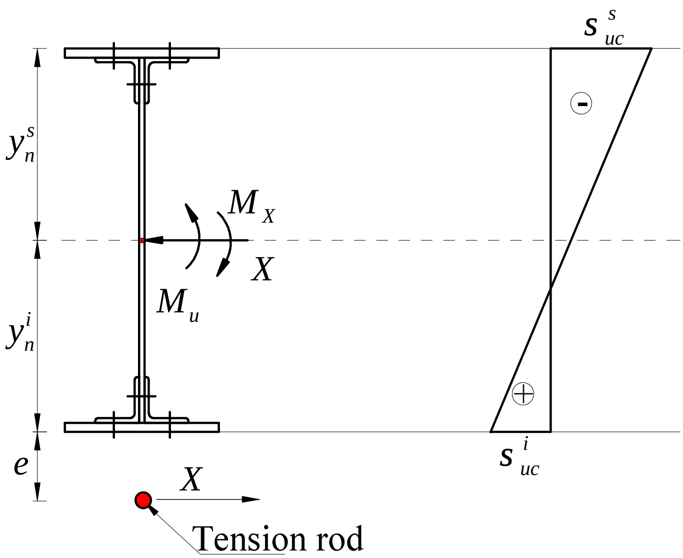

Figure 6

$$M_x = X(e + y_n^i);$$

$$s_{uc}^{s} = \frac{M_u}{I_{gn}} y_n^s + \frac{X}{A_{gn}} - \frac{M_x}{I_{gn}} y_n^s; \tag{6}$$

$$s_{uc}^{i} = \frac{M_u}{I_{gn}} y_n^i - \frac{X}{A_{gn}} - \frac{M_x}{I_{gn}} y_n^i;$$

c) The final stress state in the girder results when the two states presented are combined.

$$s_c^s = s_{gn}^s + s_{uc}^s \le s_{an};$$

$$s_c^i = s_{gn}^i + s_{uc}^i \le s_{an}; \tag{7}$$

The tension rod section is also checked out:

$$s_t = \frac{X}{A_t} \le s_a^t; \tag{8}$$

The necessary value of force X, so that the strength of the consolidated girder is ensured, is determined from the relations (7) and two values for X are resulted. The highest one is used to determine the necessary area of the consolidation tension rod from the relation:

$$X = \frac{M_{um}}{\left(e + y_n^i\right) + \dfrac{I_{gn}^b}{\left(e + y_n^i\right)}\left(\dfrac{1}{A_{gn}^b} + \dfrac{1}{A_t^b}\right)}; \tag{9}$$

The relation (9) results from the solving of the undetermined static system from Figure 7.

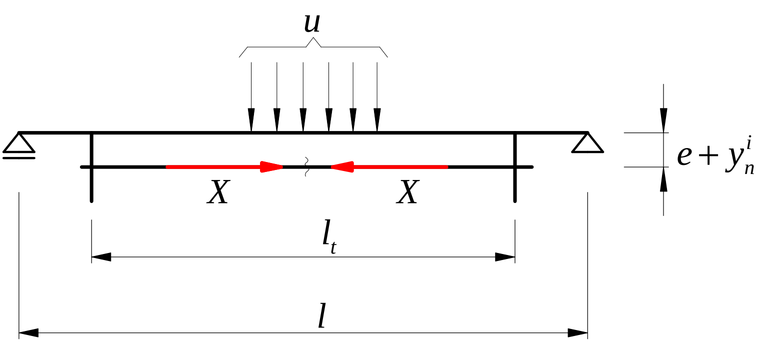

Figure 7

The relation (9) will be used for any position of the traffic load on the structure if for the bending moment produced by them on the tension rod consolidation length of the girder is considered a weighted average value Mum, in these conditions the free term ? 1p of the static balance equation:

$$Xd_{11} + \Delta_{1p} = 0$$

with an invariable form.

From relation (9) results:

$$A_{t_{nec}}^b = \frac{1}{\dfrac{\left(e + y_n^i\right)}{I_{gn}^b}\left[\dfrac{M_{um}}{X} - \left(e + y_n^i\right)\right] - \dfrac{1}{A_{gn}^b}}; \tag{10}$$

It is made up the tension rod section with the area $A_{t_{ef}}^b \geq A_{t_{nec}}^b$, it is recalculated X with relation (9) and it is checked the consolidated girder section with relation (7) and the tension rod section with relation (8).

3. CASE STUDY

The three consolidation methods are applied for main girder of a bridge with the following characteristics (Figure 8):

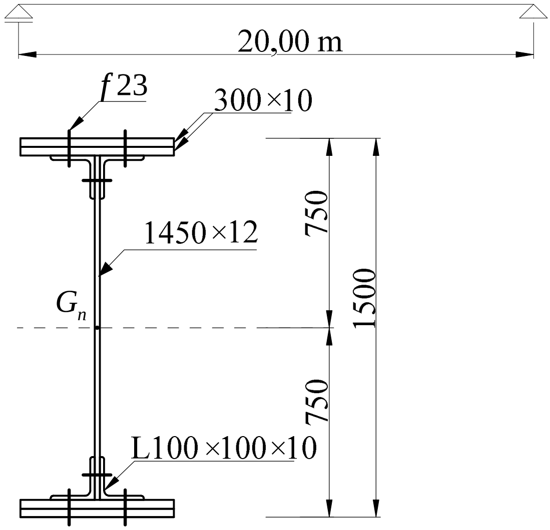

Figure 8

$$M_{gn} = 1112 \ kNm; \qquad M_u = 1440 \ kNm;$$

$$s_{an} = 150 \ N/mm^2; \qquad s_{ac} = 145 \ N/mm^2;$$

$$A^b_{gn} = 37080 \ mm^2; \qquad A_{gn} = 34320 \ mm^2;$$

$$I^b_{gn} = 1340240 \times 10^4 \ mm^4; \qquad I_{gn} = 1191138 \times 10^4 \ mm^4;$$

The maximum unit stress of the girder produced by the bending moment given by the permanent and traffic loads is up to 160,8 N/mm^2.

3.1. The consolidation by enhancing the base of the girder section with chord plates applied directly on the base of the girder without cancelling out the permanent loads stress

The inferior base of the girder is consolidated with three 350×10 mm steel plates (Figure 9) from OL 37.2 (s_{ac}=145 N/mm^2) steel.

$$M_{g'} = 41,5 \ kNm; \qquad I_{gc} = 1613178 \times 10^4 \ mm^4;$$

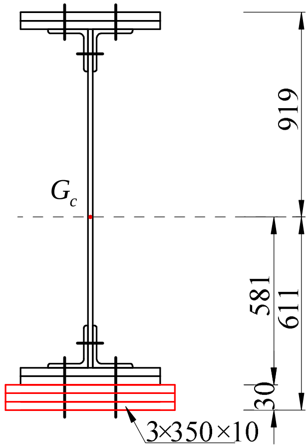

Figure 9

From relations (1), (2), (3) results:

$$s_{gn}^{s} = s_{gn}^{i} = \frac{1112\times10^{6}}{1191138\times10^{4}}\times750 = 70\,{}^{N}\!/\!_{mm^2};$$

$$s_{g'}^{s} = s_{g'}^{i} = \frac{41,5\times10^{6}}{1191138\times10^{4}}\times750 = 2,6\,{}^{N}\!/\!_{mm^2};$$

$$s_{uc}^{s} = \frac{1440\times10^{6}}{1613178\times10^{4}}\times919 = 82\,{}^{N}\!/\!_{mm^2};$$

$$s_{uc_1}^{i} = \frac{1440\times10^{6}}{1613178\times10^{4}}\times581 = 52\,{}^{N}\!/\!_{mm^2};$$

$$s_{uc_2}^{i} = \frac{1440\times10^{6}}{1613178\times10^{4}}\times611 = 54,6\,{}^{N}\!/\!_{mm^2};$$

$$s_{c}^{s} = 70,00 + 2,60 + 82,00 = 154,60\,{}^{N}\!/\!_{mm^2} \approx s_{an} + 3\% = 154,50\,{}^{N}\!/\!_{mm^2};$$

$$s_{c_1}^{i} = 70,00 + 2,60 + 52,00 = 124,60\,{}^{N}\!/\!_{mm^2} < s_{an} = 150\,{}^{N}\!/\!_{mm^2};$$

$$s_{c_2}^{i} = 54,60\,{}^{N}\!/\!_{mm^2} < s_{ac} = 145\,{}^{N}\!/\!_{mm^2};$$

It can be observed that the usage degree of the consolidation steel plates is:

$$\frac{54,60}{145}\times100 = 37,60\%$$

so an uneconomical usage of the steel plates.

The steel consumption for the consolidation is:

$$G_c = 3\times 0{,}35\times 0{,}01\times 7850\times 12{,}00 = 990\ kg$$

if the consolidation is made on the central area of the main girders with a 12 m length.

The length on which the girder is consolidated is established from the following condition: the unconsolidated sections from the ends of the consolidation elements should be able to take over the bending moment given by the traffic loads.

3.2. The consolidation by enhancing the base of the girder section with chord plates applied directly on the base of the girder by cancelling out the permanent loads stress

The inferior base of the girder is consolidated with two 300×10 mm steel plates (Figure 10) from OL 37.2 (s $_{ac}$=145 N/mm^2) steel.

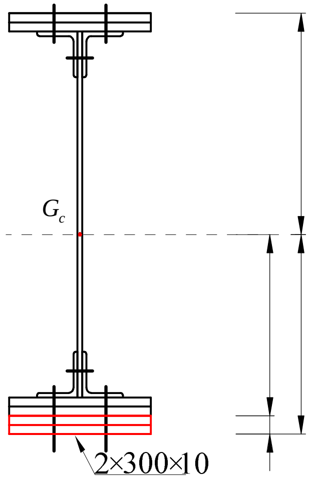

Figure 10

$$M_{g'} = 24{,}00\ kNm; \qquad I_{gc} = 1446893\times 10^4\ mm^4;$$

From relation (4) results the following stress state:

$$s_c^s = \frac{1112\times 10^6 + 24\times 10^6 + 1440\times 10^6}{1446893\times 10^4}\times 856 = 152{,}40\ N/mm^2;$$

$$s_c^s = 152{,}40\ N/mm^2 < s_{an} + 3\% = 154{,}50\ N/mm^2;$$

$$s_{c_1}^{i} = \frac{1112\times10^{6} + 24\times10^{6} + 1440\times10^{6}}{1446893\times10^{4}}\times644 = 114,7\,{}^{N}\!/_{mm^2};$$

$$s_{c_1}^{i} = 114,7\,{}^{N}\!/_{mm^2} < s_{an} = 150\,{}^{N}\!/_{mm^2};$$

$$s_{c_2}^{i} = \frac{1112\times10^{6} + 24\times10^{6} + 1440\times10^{6}}{1446893\times10^{4}}\times664 = 118,3\,{}^{N}\!/_{mm^2};$$

$$s_{c_2}^{i} = 118,3\,{}^{N}\!/_{mm^2} < s_{ac} = 145\,{}^{N}\!/_{mm^2};$$

It can be observed that the usage degree of the consolidation steel plates is:

$$\frac{118,3}{145}\times100 = 81,50\%$$

so an efficient usage of the steel plates.

The steel consumption for the consolidation is:

$$G_c = 2\times0,30\times0,01\times7850\times12,00 = 565\ kg$$

3.3 The consolidation with unprestressed rigid steel tension rod applied under the inferior base of the main girders

The inferior base of the girder is consolidated with unprestressed rigid steel tension rod consisting of two L-shaped bars from OL 37.2 (s ac=145 N/mm2) steel, located under the girder base at a distance of e = 300 mm (Figure 11).

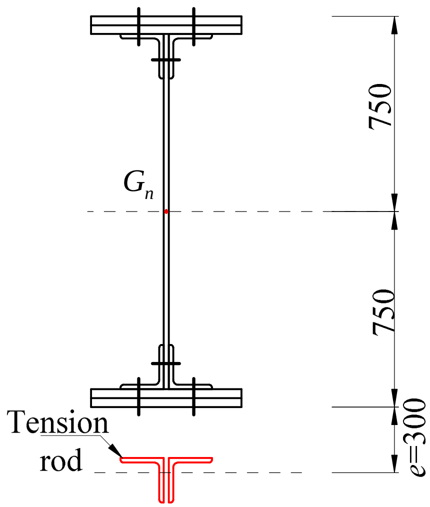

Figure 11

$$M_{um} = 1120,00 \ kNm;$$

The length on which the girder is consolidated (the length of the consolidation tension rod) is lt = 12.00 m.

On the unconsolidated section of the girder results the stress state given by the permanent loads illustrated by relation (5).

$$s_{gn}^{s} = s_{gn}^{i} = \frac{1112\times 10^{6}}{1191138\times 10^{4}}\times 750 = 70 \, {}^{N}\!/_{mm^{2}};$$

By applying condition (7) for the total unit stress of the consolidated girder, in which s_{uc}^{s} and s_{uc}^{i} are given by relation (6), results the axial tension force needed in the tension rod (two values are obtained, from which the highest value is taken into consideration).

$$X = 292,09 \ kN$$

By replacing the value of X, as it is mentioned previously, in relation (10) results the necessary rough area of the tension rod:

$$A_{t_{nec}}^{b} = \frac{1}{\frac{(300+750)}{1340240\times 10^{4}}\left[\frac{1120\times 10^{6}}{292,09\times 10^{3}} - (300+750)\right] - \frac{1}{37080}} = 5231 mm^{2};$$

For the tension rod section are selected two L-shaped bars 2L 100×100×12 for which:

$$A_{t_{ef}}^{b} = 2\times 2750 = 5500 mm^{2};$$

X is recalculated with relation (9):

$$X_{ef} = \frac{1120\times 10^{6}}{(300+750) + \frac{1340240\times 10^{4}}{(300+750)}\left(\frac{1}{37080} + \frac{1}{5500}\right)} = 301480 \ N;$$

The unit stress of the consolidated girder is verified with relations (7) in which are introduced relations (5) and (6), resulting:

$$s_{c}^{s} = \frac{1112\times 10^{6}}{1191138\times 10^{4}}\times 750 + \frac{1440\times 10^{6}}{1191138\times 10^{4}}\times 750 + \frac{301480}{34320} +$$
$$+ \frac{301480(300+750)}{1191138\times 10^{4}}\times 750 = 149,70 \, {}^{N}\!/_{mm^{2}} < s_{an} = 150 \, {}^{N}\!/_{mm^{2}};$$

$$\sigma_c^i = \frac{1112\times10^6}{1191138\times10^4}\times750+\frac{1440\times10^6}{1191138\times10^4}\times750-\frac{301480}{34320}-$$
$$-\frac{301480(300+750)}{1191138\times10^4}\times750=132,10\, {}^{N}\!/_{mm^2} < \sigma_{an} = 150\, {}^{N}\!/_{mm^2};$$

The tension rod is checked at the tensile axial force (the net section of the tension rod has been taken into account) with relation (8):

$$A_{t_{ef}} = 5500-2\times23\times12 = 4948 mm^2$$

$$\sigma_t = \frac{X_{ef}}{A_{t_{ef}}} = \frac{301480}{4948} = 61,00\, {}^{N}\!/_{mm^2} < \sigma_a^t = 145\, {}^{N}\!/_{mm^2};$$

The usage degree of the tension rod is:

$$\frac{61}{145}\times100 = 42\%$$

The steel consumption for the consolidation is:

$$G_c = 2\times21,60\times12,00 = 519\ kg$$

If the tension rod is placed at a distance of e = 50 cm results an axial force of X = 217880 N, and the tension rod section will consist of two L-shaped bars 2L 80×80×10 with $A_{t_{ef}}^b = 3020 mm^2$.

4. CONCLUSIONS

If the results obtained through the consolidation methods discussed are analysed the following conclusions can be drawn:

1. The consolidation through these three methods does not use initial stress states obtained through different means like pre-bending the structure before consolidation, using some pre-stressed consolidation elements;
2. The consolidation solution presented at 2.2 offers more advantages than the one presented at 2.1 as a result of cancelling out the permanent loads during the consolidation, the steel consumption needed for the consolidation being lower.
3. The tension rod consolidation is the most advantageous because the steel consumption needed for the consolidation is the lowest; the disadvantage is that the bridge building height increases, and the bridge outlet decreases.

Acknowledgements

Notation:

- $y_n^s; y_n^i$ -the distance from the section centroid of the unconsolidated girder section to the top fibre/bottom fibre;
- $y_c^s; y_c^i$ -the distance from the section centroid of the consolidated girder section to the top fibre/bottom fibre;
- G_n -the section centroid of the unconsolidated girder section;
- G_c -the section centroid of the consolidated girder section;
- Δt -the thickness of the consolidation chord plates applied on the base of the girder section;
- l_t -the length of the consolidation tension rod;
- e -the distance from the section centroid of the consolidation steel tension rod to the inferior base of the girder;
- $I_{gn}; I_{gn}^b$ -the moment of inertia (second moment of area) of the unconsolidated net/rough girder section;
- I_{gc} -the moment of inertia (second moment of area) of the consolidated net girder section;
- $A_{gn}; A_{gn}^b$ -the area of the unconsolidated net/rough girder section;
- $A_t; A_t^b$ -the net/rough area of the consolidation tension rod;
- M_{gn} -the maximum bending moment given by the weight of the unconsolidated structure;
- $M_{g'}$ -the maximum bending moment given by the weight of the consolidation elements;
- M_u -the maximum bending moment given by the traffic loads;
- M_{um} -the weighted average value of the bending moment M_u on the tension rod consolidation length;
- X -the axial stress from the consolidation tension rod;
- M_X -the girder bending moment given by the axial stress X;
- $s_{gn}^s; s_{gn}^i$ -the normal unit stress produced by M_{gn} on the unconsolidated girder section at the top fibre/bottom fibre;
- $s_{g'}^s; s_{g'}^i$ -the normal unit stress produced by $M_{g'}$ on the unconsolidated girder section at the top fibre/bottom fibre;

- $s_{uc}^{s}; s_{uc}^{i} (s_{uc_1}^{i}; s_{uc_2}^{i})$ -the normal unit stress produced by M_u on the consolidated girder section at the top fibre/bottom fibre (in the points 1 and 2);
- $s_{c}^{s}; s_{c}^{i} (s_{c_1}^{i}; s_{c_2}^{i})$ -the total unit stress at the top fibre/bottom fibre (in the points 1 and 2) of the consolidated girder;
- s_{t} -the unit stress in the consolidation tension rod;
- s_{an} -allowable normal stress of the steel form the unconsolidated girder;
- s_{ac} -allowable normal stress of the steel form the consolidation elements;
- s_{a}^{t} -allowable normal stress of the steel form the consolidation tension rod;

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The Dimensioning of the Rigid Runway Structures

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Abstract

The article presents the dimensioning and design methods for airport runway systems (new/ reinforced) and roads elaborated in Romania.

There are emphasized the design solutions by the means of the computational schemes (using FEM) the hypothesis and computational parameters.

Finally, there are depicted several design diagrams.

KEYWORDS: Rigid runway structure, runway, airport structure, reinforcement, computational scheme, dimensioning diagram

1. INTRODUCTION

During the last decade of the past century the dimensioning method of the rigid runway structures – P.D-177-76 – represented the adaptation of the Soiuzdornâi method to the Romanian conditions; this method had been provided not only some limitations concerning the scheme/ relationship/ computational parameters (the initial Westergaard relationship in the Mednicov format, the runway cement concrete characterized through the mark, the characteristic of the linear deflection environment that sustained the plate, represented by the deflection modulus, the fatigue law expressed as a function of the equivalent traffic) but also the disadvantage of some laborious .interpolations.

For the dimensioning of the rigid airport runway structures there wasn't a self method, one used for instance the Pickett-Ray method [6].

This paperwork presents – in a synthetic way- the elements of the dimensioning methods carried on in the frame of a grant with INCERTRANS Bucharest, both for the airport rigid runway structures, new (A)/ reinforced (B) and new runways (C).

2. COMMON FEATURES OF THE CREATED DIMENSIONING MEHODS

2.1. The Computational Scheme

2.1.1. The computational scheme is made through the FEM (the Finite Element Method), the multilayer approach, correlated with the symmetry.

Tri- dimensional finite elements are used (solid –Brick, parallelepiped) with 8 joints. Each joint is granted with 3 DOFs i.e. the three direction translations.

The models are made in the linear-elastic domain, with isoparametric elements of 2nd order integration.

The level of the horizontal mesh assures the optimization of the element number, further refinements leading to variations of maximum 0,5% of the results.

A mesh refinement was carried on in the traces in order to tune precisely the structural response.

The use of successive layers is justified by the vertical stress distribution; the pressures onto the lane were transmitted to the finite elements.

The dimensions of the successive layers allows the optimization of the finite element dimension rations: the number/thickness of the layers pays respect to the condition of stress compatibility (a further model, supplementary refined, didn't lead to a variation of the stress distribution larger than 0,5% in the vertical plane).

The model presents geometric, elastic and mechanic symmetry in several situations; therefore the hemi-structures were used instead (Fig. 1).

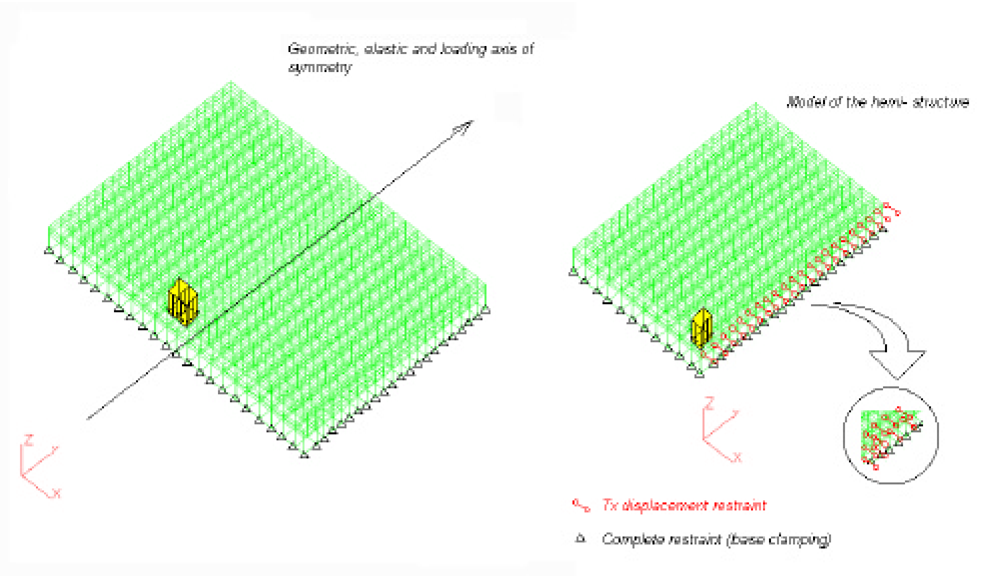

Figure no. 1

2.1.2. The models were built by assembling the substructures specific to the various runway structures (new structures/ reinforced).

In all cases (A… C), the "basic" substructure is made of a single layer of finite elements. The layer is equivalent to the sub- adjacent layers of the runway cement concrete plate; it has a 40cm thickness and the dynamic modulus of elasticity (E) is obtained using the (2.1) transform:

$$E = K_o \cdot (h = 40\text{cm})\,[\text{MN/m}^3] \tag{2.1}$$

The reaction modulus at the surface of the equivalent layer is obtained from the diagrams, as a function of the equivalent thicknesses of the sub- adjacent layers, which are computed by the means of an AASHTO Road Test/ SBA-STBA relationship.

2.1.3. For the (A) and (C) cases the model is composted of two substructures; the upper one is represented by the runway cement concrete plate; this was made of 2… 4 layers of BRICK elements, depending on the thickness.

2.1.4. For the (B) case, beyond the “basic” substructure, the modeling process takes into account the following layers:

(a) The hypothesis of non- adherent plates: the reinforcing plate/ the intermediate layer made of asphalt mixture/ the existent plate.

(b) The hypothesis of partially adherent plates: the reinforcing plate/ the connection in-between plates/ the existent plate.

The thickness of the intermediate layer of asphalt mixture was stated following a stress/ strain analysis; after analyzing the study it resulted a stress increase in the reinforcing slab proportional with the layer thickness; this variation is stabilized for a 5 cm thickness.

The “intermediate layer of asphalt mixture” substructure is modeled by the means of a single layer of BRICK elements.

The reinforcing slab substructure is made of 2… 4 layers of BRICK elements, depending on the thickness.

2.1.5. The mathematical models were run, tested, validated and applied using the ALGOR Software for the structural analysis based on the Finite Element Method.

2.2. The deflection characteristic, for all the constituent materials is represented by the dynamic modulus of elasticity (E- MPa).

The deflection characteristic for the foundation ground is the reaction modulus (K-MN/m3).

The runway cement concrete (BcR) is characterized by the means of classes; the class defines the characteristic bending tensile strengths (Rkinc150) at 28 days.

An unique value of E=30,000MPa was adopted for the BcR in order to carry out the dimensioning diagrams; this value is motivated by the need to limit the number of the diagrams (under the circumstances of a limited influence of E [5]).

2.3. The dimensioning criterion is defined by the (2.2) condition:

$$s_{t} \leq s_{t-adm} \;\; [\mathrm{MPa}] \qquad (2.2)$$

Where:

σ_t - the bending tensile stress in the plate;
$\sigma_{t\text{-}adm}$ - the limit bending tensile stress of the BcR.

2.3.1. In the “A” case:

$$s_{t-adm} = \frac{R^{K}_{inc.150} \cdot a}{C_S} \;\; [\mathrm{MPa}] \qquad (2.3)$$

Where:

α - Coefficient that takes into account the increase of the concrete strength in the range 28… 90 days; α=1,1;
C_S – safety factor is a function of the transfer devices and the unpropitious geotechnical, climatic and traffic conditions. The values for CS = 1,8 and C = 2,6 are similar to those of the French Codes [5].

2.3.2. In the "B" case:

$$s_{t-adm} = \frac{R^K_{inc.150} \cdot a}{C_S} \cdot C_{SS} \text{ [MPa]} \tag{2.4}$$

Where:

α, C_S – idem case "A";
C_{SS} – coefficient of the structural state for the slab that is to be reinforced:

C_{SS} = 0,35- very damaged (cracked) plates with damaged gaps;
C_{SS} = 0,75 - cracked plates but without generalized damages; slabs with broken corners or with some structural cracks;
C_{SS} = 1,00 - slabs in good condition, without structural damages; for this kind of plates the reinforcing is imposed by the outrunning of the traffic foreseeing.

2.3.3. In the "A" and "B" cases, the fatigue law (eq. no. 2.5) is taken into account to establish the regular load for the computation

$$C = 1{,}2 - 0{,}2 \cdot \log N \tag{2.5}$$

2.3.4. In the "C" case:

$$s_{t-adm} = R^K_{inc.150} \cdot a \cdot ß \cdot (1 - ? \cdot \log N) \text{ [MPa]} \tag{2.6}$$

Where:

α – Idem, case "A";
β – Coefficient that takes into account the increasing value of the concrete strength in time; β=1,1. If is absent in the owner studies, one may use it in the case of local roads.
N – The computational traffic (m.o.s.); OS115 KN.
γ = 1/14.

σ_{t-adm} is to be referred to the lower layer, the resistant layer, in the case of the plate made of 2 layers because of the economic reasons.

2.4. The elliptical prints from the "A" case – the standard hemi- axle (57,5 KN) and from the "B" and "C" cases (simple, twin, boogie and tandem landing gears) are transformed into rectangular prints for the computational model (Fig. no. 2).

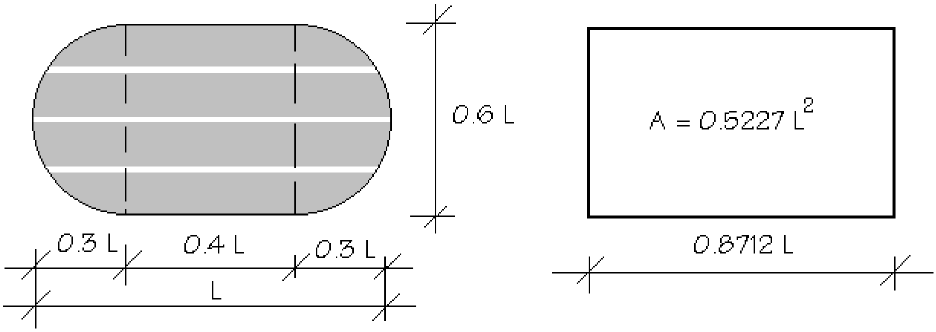

Fig. 2 The real gear print and the computational print

2.5. The loading position: the print/ prints are tangent to the longest edge of the slab, in the conditions of the uniformly restrained slab.

In the Figure no. 3 it is depicted an example of the dual and boogie print positions.

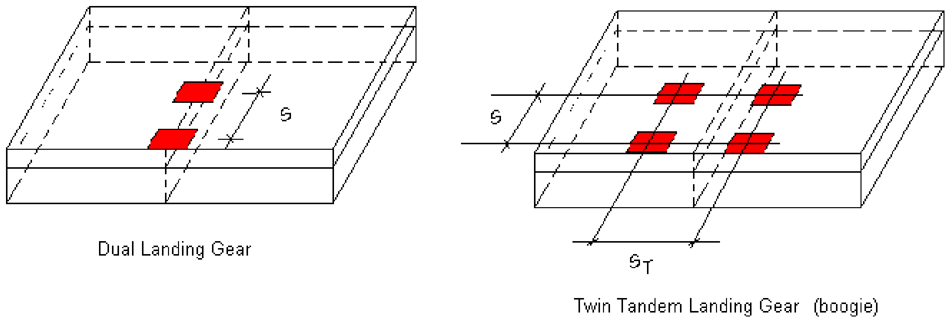

Fig. 3 The 2nd load position

2.6. Geometric Elements

In order to limit the number of the dimensioning diagrams, it was selected a number of representative plates (by taking into account the plane dimensions), for each case (A… C), as follows:

(A) The 7,00 x 5,00m slab, legitimated by:

- the working span of the concrete spreading machine;
- the maximum allowable distance between the gaps.

(B) The 3,75 x 5,00m slab (the existent plate that has to be reinforced/ it imposes the gap positions of the reinforcing slab). For the range (3,00 x 5,00m) … (5,00 x 5,00m), the reinforcing thicknesses are maintained between the limits ± 1,0cm (the degree of precision assured by the use of the diagrams). For particular dimensions (for instance 3,00 x 6,00m) the differences are significant and require individual project; in this case the making of the diagrams is not justified.

(C) The (5,00… 6,00m) x (3,25… 4,25m) slab; the slab width variation domain assures the correlation with the technical class (I… V)/ the economic category of the road (highway… local road).

3. THE SPECIFIC ITEMS

3.1. In the cases "A" and "B": the SBA/STBA [5] methodology is considered for:

- the geometric characteristics (gauge, wheelbase) of the standard landing gear (see. Table No.1);
- the pounded real load, as a function of the type/ destination of the airport surface (landing/ take-off runway/ run path/ fast come off; stop prolongation – SWY, platforms);
- the normal computation load, that takes into account the superposition of the prints in case of several motions of the aircrafts (landing/ take-off/ run);
- the general dimensioning method for the standard aircraft;
- the optimized dimensioning method.

Table 1 The features of the standard landing gears

Landing Gear Type	Gauge (s) - cm	Wheelbase (Sp) - cm
Twin	70	-
Boogie	75	140

3.2. A computer software was developed, based on the AASHTO methodology [7]; the software optimizes the division of the structures into sectors that have to be reinforced. The program allows the definition of several criteria (the damaging *indexes, C_{SS}, the FWD/ HWD deflections, the type/ characteristics of the foundation ground/ foundation layers).*

3.3. The dimensioning diagrams are drawn for the following domains:

3.3.1. (A): Simple gear: P = 5…40 tf.
Twin gear: P = 7,5…42,5tf.
Boogie gear: P = 15… 105tf.
Tandem gear: P = 25… 37,7tf.

In the Figure No.4 it is depicted an example of a boogie landing gear (P = 45… 75tf.)

3.3.1. (B): Twin gear: P = 27,5…42,5tf.
Boogie gear: P = 60… 105tf.

The diagrams are drawn in the following hypothesis:
Partially adherent slabs: C_{SS} = 0,75; 1,00.
Non- adherent slabs: C_{SS} = 0,35; 0,75.

The minimum thickness of the reinforcing slab is of 15cm but the recommended value is 18cm.

The figure no. 5 presents the diagram for the twin gear, P = 42,5tf, in the assumption of the non- adherent slabs.

3.3.3. (C) The dimensioning diagrams are taking into account the computational assumptions regarding the load combination (from traffic - σ_t; from the daily temperature gradient - $\sigma_{t\Delta t}$); these assumptions are correlated with the technical class of the runway:

$$\sigma = \sigma_t + \mu \cdot \nu \cdot \sigma_{t\Delta t} \leq \sigma_{tadm} \ \text{[MPa]} \tag{3.1}$$

Technical class I, II: μ = 0,8; ν = 1,0;

Technical class III, IV: μ = 0,8; ν = 0,65;

Technical class V: μ = 0; ν = 0.

In the figure no. 6 it is presented the diagram for the runways from the technical class I, II.

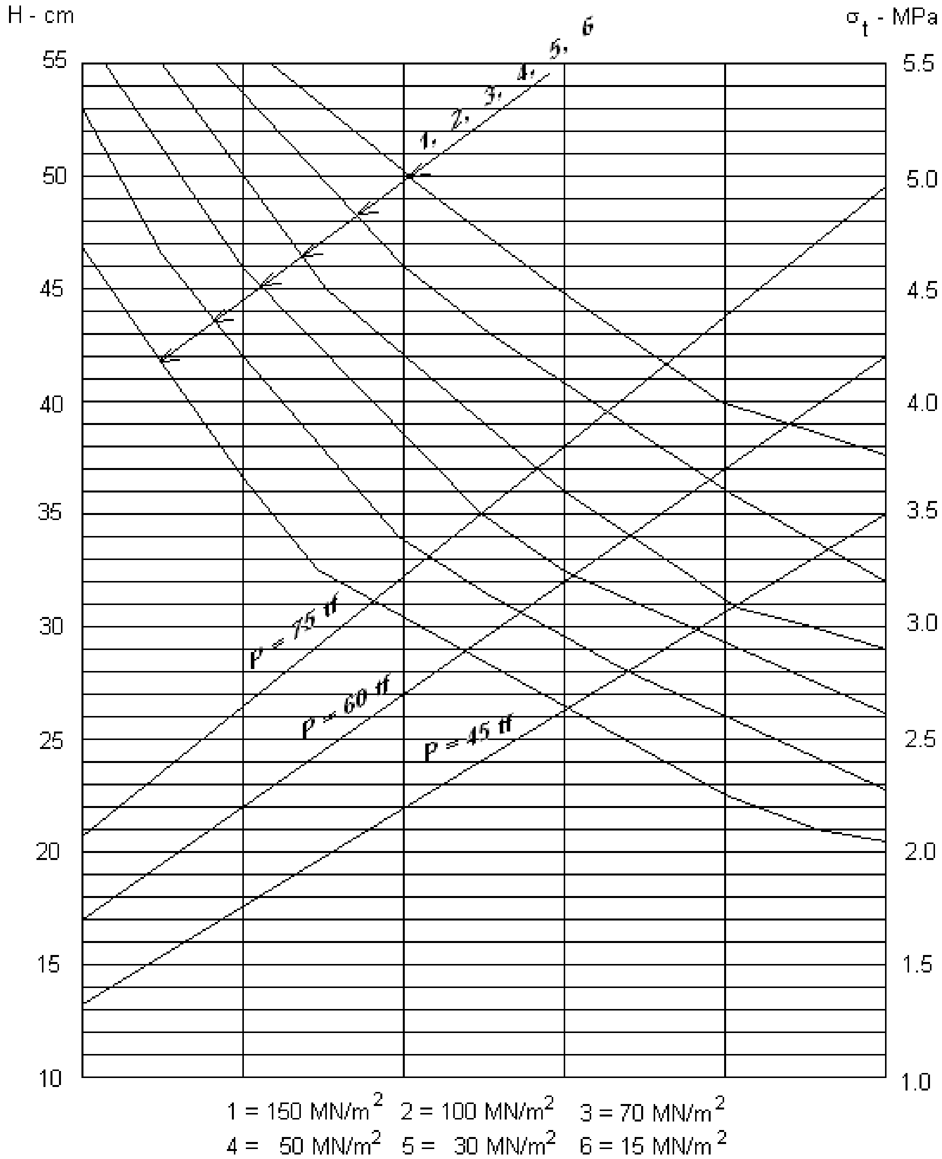

Fig. 4 The twin tandem landing gear P = 45, 60, 75tf.

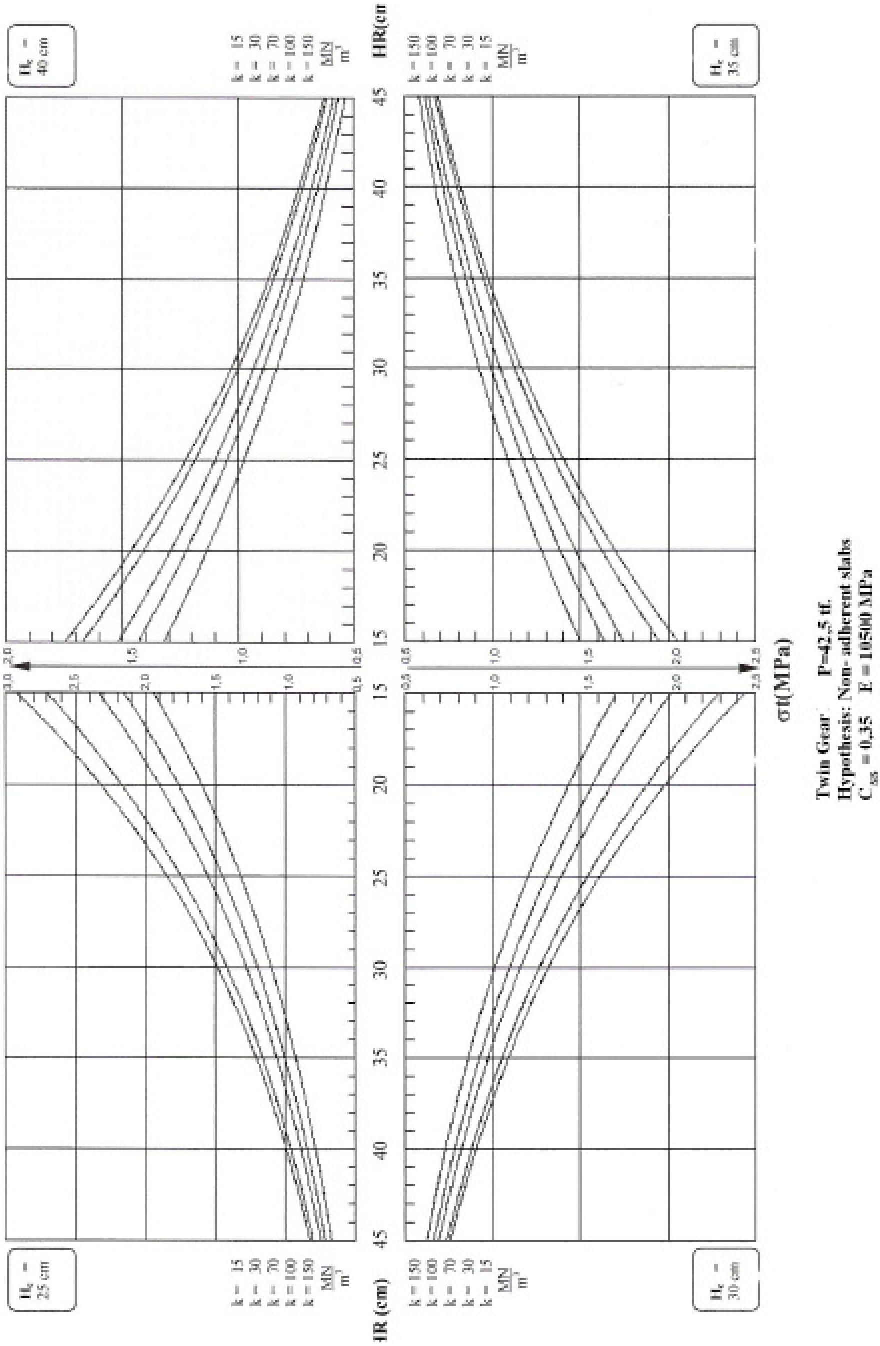

Fig. 5 The twin landing gear P = 42,5tf.

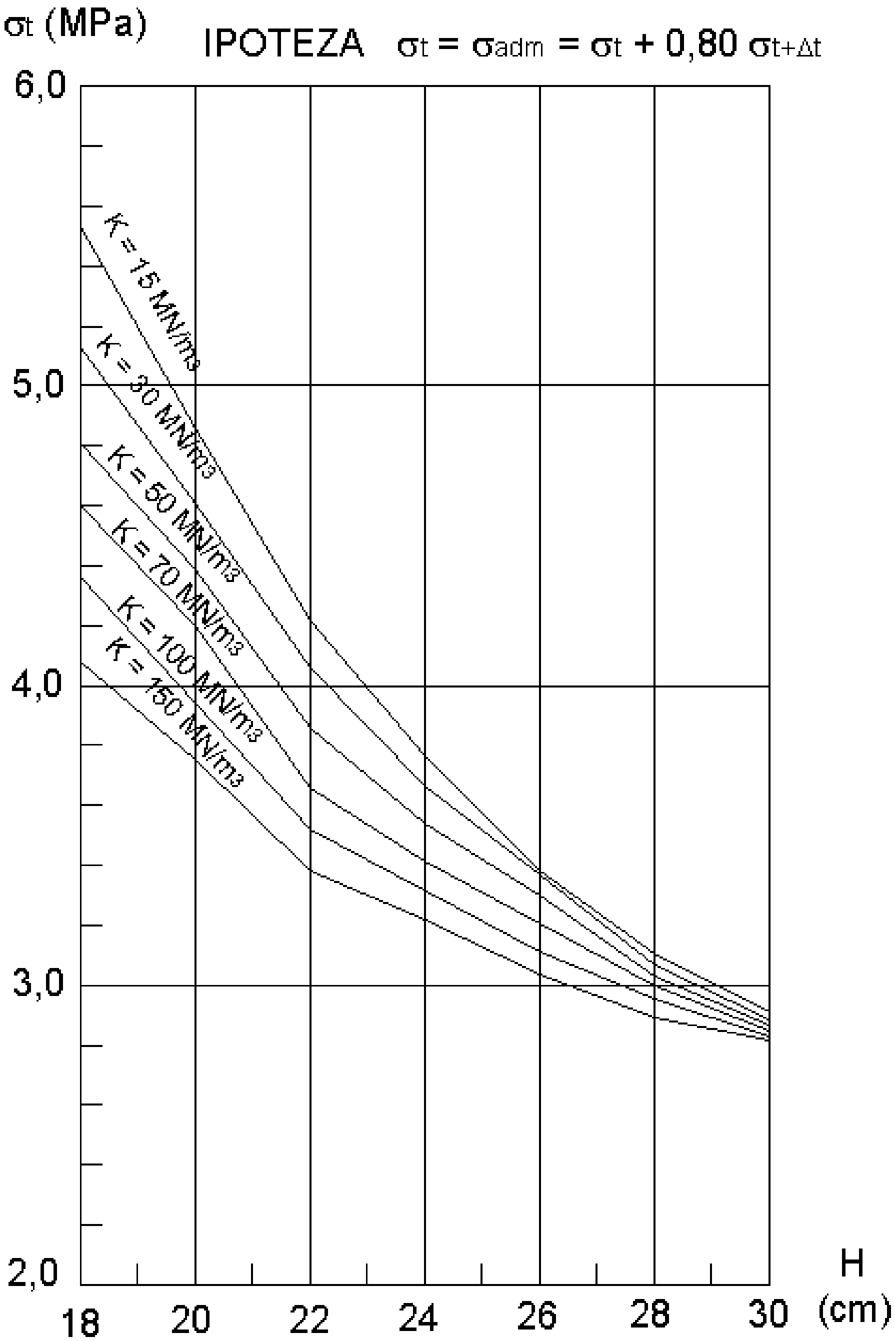

Fig. 6 The diagram for runways from the technical class I, II

3.3.4. The 4th degree polynomial correlations are allowing the increase of the computational precision for the slab thickness.

4. CONCLUSIONS

For all the classes of rigid runway structures – new airport runway/ reinforced, new runways – the computational schemes are carried on by the means of FEM.

The computational scheme for the new structures of roads is taken – in the IPTANA Bucharest format – also for the reinforcing of the simple runway structures/ hemi- rigid with runway cement concrete [4].

The slab thicknesses are derived from diagrams carried on for slabs that have representative plane dimensions with respect to the classes of the runway structures/ current building conditions. The 4th order polynomial correlations are certifying the increase of the computational precision.

In case of particular plane dimensions of the slab the individual project is required.

The rigid runway structures are checked to the frost/thaw action, according to the provisions of the Romanian Code STAS 1709-90.

The items presented in this paperwork are included in the final drafts of the dedicated Design Codes [1, 2, 3] by INCERTRANS Bucharest.

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Environmental assessment of transport pavements

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Summary

The road infrastructure have a major environmental impact by covering important land surfaces and consuming very big amounts of materials from lithosphere for execution and maintenance. The impact of transport pavements needs to be considered from the design stage and to be examined during the whole life cycle of the infrastructure: raw materials extraction and initial transformation, manufacturing, placement on site, exploitation, maintenance, and demolition.

The paper examines the main environmental impacts of transport pavements and presents several methods of their environmental impact assessment, especially the LCA models, such as SimaPro, PaLATE, ROAD-RES, and EcoConcrete. A case-study using EcoConcrete LCA model is also presented.

KEYWORDS: road, transport pavements, life-cycle assessment, environmental impact.

1. INTRODUCTION

The greatest strength of road transport is its capacity to carry goods and passengers all over Europe with unequalled flexibility and at a low price. But despite the 10 hectares of new roads that are built every day in Europe, road congestion remains a serious problem, especially in urban areas (the congestion costs represent 2% of EU GDP). The associated stop-start driving means higher emissions of pollutants and greater energy consumption. The FP6 Work Programme for the Sustainable Surface Transport calls for reconciling increasing demands for personal mobility, safety and freight transport and issues of sustainability, such as the environmental impact of emissions and noise, recycling and the supply of energy and fuel. Under FP7, the Commission is proposing a "greener", "safer" and "smarter" pan-European transport system, supported by a research budget of over €4 billion in seven years [1].

Transport accounts for 32% of Europe's energy consumption and 28% of total CO_2 emissions. However, it is expected to account for 90% of the forecast increase in CO_2 emissions between 1990 and 2010. Without radical change, road transport will be a main reason for Europe's failure to meet its Kyoto commitments [2].

Transport has a leading environmental impact, as much through the infrastructure quotient as through the superstructure. The infrastructure covers important land surfaces, affecting the ecosystems, and consumes the very big amounts of materials from lithosphere for execution and maintenance. The superstructure (vehicles) consumes hydrocarbons (fossil fuels) and represents an important source of chemical pollution.

The impact of road infrastructure needs to be considered from the design stage to reduce the visual impact of major roads, protect natural habitats and control water run-off, and effectively accommodate low speed and non-motorised transport. According to the European Road Transport Research Advisory Council [3] the following aspects are important: the consideration of life cycle assessment for maintenance and construction materials, further research on noise mitigation, new land planning policies, new infrastructure design concepts for the preservation of natural habitats, the improvement of existing traffic management systems to smooth traffic flow and thus reduce emissions and noise, further study on the impact of road transport on water quality using life assessment to evaluate direct emissions from roads and vehicles, and indirect effects such as the production of biofuels. Further improvements in road construction and demolition waste recycling technologies are needed to increase the percentages of recycled material that can be used in production of new high quality pavement surface layers.

This paper will briefly examine the issues relating the environmental impact of road pavements during the main stages of their life cycle, from the raw materials extraction to the exploitation of the infrastructure. In addition, several of the most utilised methods for environmental impact assessment will be presented, including a case-study using EcoConcrete LCA model.

2. ENVIRONMENTAL IMPACT OF TRANSPORT PAVEMENTS

The location, construction, exploitation and maintenance of linear infrastructures for transport have an important environmental impact, mainly materialized in:

- occupancy of land surfaces and the affection of ecosystems,
- consumption of large amounts of materials from lithosphere,
- use of technologies that are fossil energy consuming and polluting.

Environmental impact of transport pavements has to be considered for each life cycle stage: (a) raw materials extraction and initial transformation, (b) manufacturing, (c) placement on site, (d) use and maintenance, (e) removal, recycling and disposal.

The infrastructure's locations affect the conditions of environment through**:** land occupancy, disruption of existing conditions and ecological equilibrium, and change of landscape.

The aggregates of ballast pits represent a non-renewable natural resource. Furthermore, the exploitation of this resource has effects on the environment, in the form of:

- degradation of the landscape,
- location of activity in the minor river beds of the waters (the extraction is carried out under water) or terrestrial,
- use of technological equipment that generates chemical pollution, noise and vibrations,
- transport of aggregates within the perimeter of the exploitation and up to users.

The quarries of massive rocks exploitation represent sources of dust, strong vibrations and intermittent noises, resulting from the use of explosives as well as from the aggregates processing units (breaking machines, sieves). The level of noise ranges from 70-80 dB(A) at a distance of 50 m from its perimeter and to 55-60 dB(A), at a distance of 400 m [4]. A higher level of pollution is produced by the technological transports and the transports towards users; in other words, a supplementary level of pollution is due to the emissions of gases, noise and dust from the transportation of aggregates within the quarry and up to users.

Construction sites have an impact on the natural and anthropic factors of environment, consisting in:

- temporary occupancy and division of field surfaces,
- chemical and energetic pollution, through noise and vibrations,

- disruption of neighbours’ life, communications and economical activities,
- disruption of fauna and flora,
- off-road degradation through the location of borrow areas (i.e. where material has been dug for use at another location) and/ or of the provisional or definitive deposits,
- existing roads degradation by taking over the transports generated by construction sites.

The excavations and fillings, required for cuttings and backfills, respectively, affect the earth layers and the regime of underground water from that zone.

The semi-product preparation for road pavements (asphalt mixtures and Portland concretes) causes chemical and energy pollution (with noise and vibrations).

The asphalt mixing plants, which produce asphalt mixtures using the hot mix method, have an impact on:

- soil through the occupancy of land surfaces, accidental effluents of oil and fuels on soils, deposits of solid aerosols discharged together with the burned gases,
- the phreatic or surface waters in that area,
- the atmosphere with the pollutants that are emitted through the chimney of the plant deriving mainly from the burning of the fuels, whose amount and composition depend on the type of fuel, capacity of production of the asphalt mix plant, humidity of the aggregates, quality of adjustment and development of combustion. Due to the wind action, the maximum concentration to the ground level of the dispersed pollutant (imission) is found out at distances of 15 to 30 times the height of the stack.
- the level of noise (at source exceeds 85 dB(A)).

The Portland concrete mixing plants cause an impact:

- on atmosphere through the pollution with mineral dusts from the manipulation of aggregates, cement and, possibly, of the thermal power-station ash (whenever this is utilised),
- on water through the discharge of the circulating water of the mixing plant and of the concrete transportation vehicles,
- due to the noise and vibrations produced by the equipment used for loading aggregates in bunkers, conveyer belts, the mixer and vehicles transporting the concrete to the construction site,
- associated chiefly with their location in the urban landscape.

Impact is also caused to the existing road network, in the vicinity of the construction site, due to the supplementary traffic, generated by the construction site. This includes disruption of the neighbours and current traffic, and the supplementary loading of the road structure through the heavy duty traffic on the construction site.

The exploitation of the infrastructure represents a source of chemical pollution, with noise and vibrations of the environment.

3. METHODS OF ENVIRONMENTAL IMPACT ASSESSMENT

The number of tools and methods for environmental assessment is increasing and users of environmental information face the problem of how to interpret this information and select the most appropriate approach for a specific situation.

This chapter gives a brief, general description of several approaches that could be used in the environmental assessment of transport pavements.

3.1. Life cycle assessment

Life Cycle Assessment (LCA) is a method for analysing and assessing the environmental impact of a material, product or service throughout its entire life cycle, usually from the acquisition of raw materials to final disposal. Traditionally, the main focus in LCA has been on regional and global environmental impacts on the external environment. According to the ISO 14040, the general categories of environmental impacts to be considered in an LCA include

- resource use,
- human health,
- ecological consequences.

An LCA study consists of four steps (figure 1):

- Defining the goal and scope of the study.
- Making a model of the product life cycle with all the environmental inputs and outputs. This data collection effort is usually referred to as the life cycle inventory (LCI) stage.
- Understanding the environmental relevance of all inputs and outputs; this is referred to as the life cycle impact assessment (LCIA) phase.
- The interpretation of the study.

LCA is very complex and time consuming, because the "cradle-to-grave" life cycle always involves numerous stages and activities that give rise to a number of different environmental loadings. To keep the amount of work within reasonable bounds, the assessments must always be limited and efforts must be made to identify the critical stages of the life cycle and those factors responsible for environmental loadings. This requires not only adherence to the basic principles of LCA but also knowledge of the product or activity in question. Great effort to develop new LCA methodologies and software has been made in the last two decades.

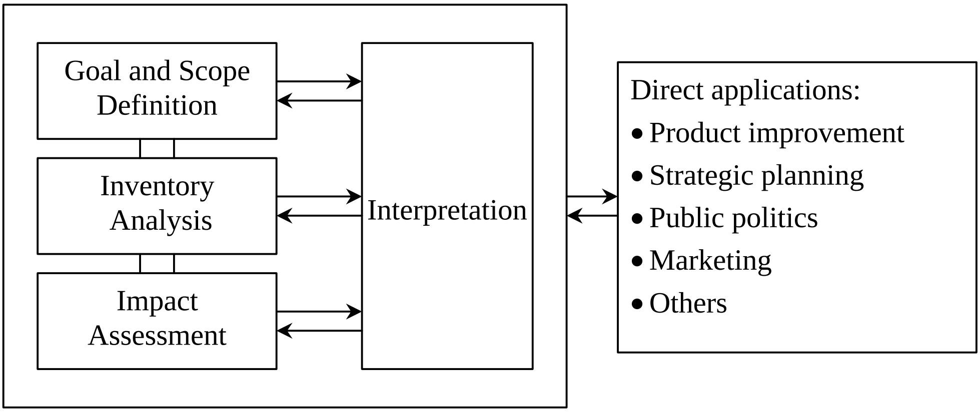

Figure 1. Phases of an LCA

One of the most widely used LCA methods is SimaPro, developed by the PRe Consultants company [5]. SimaPro software contains a number of impact assessment methods, which are used to calculate impact assessment results such as: CML 1992, Eco-indicator 95, Ecopoints 97, Eco-indicator 99.

In the transport field, Prof. Arpad Horvath from the University of California developed an Excel-based tool for LCA of environmental and economic effects of pavements and roads [6]. PaLATE is a life-cycle analysis tool that draws on environmental and economic information to evaluate the use of different materials, including recycled materials, in the construction and maintenance of pavements. PaLATE estimates energy consumption and emissions of CO_2, NO_x, PM_{10}, SO_2, CO, and informs average leachate releases for different construction materials. Environmental effects for initial construction, maintenance, and total are reported in bar graphs, and for each phase, effects from processes, material transportation, and materials production are reported separately. The user defines the design of the pavement, which results in a given type and volume of construction materials, a given combination of construction activities, and a set of prescribed maintenance activities. The life cycle of the pavements was considered as shown in figure 2. There is no end-of-life for pavements, but the maintenance box may represent major reconstructions of a road’s section that replaces the previous structure in that place.

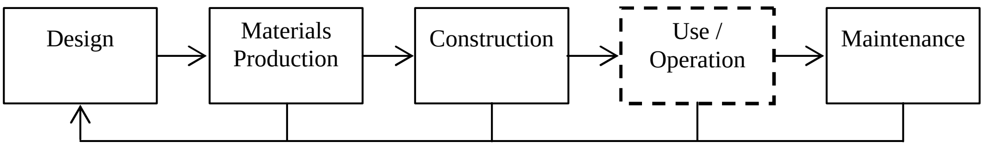

Figure 2. Life cycle phases of pavements

Another LCA methodology adapted to meet the requirements of road construction was developed by the Finnish National Road Administration [7]. The analysis includes all the significant life cycle stages covering the production and transportation of materials, their placement in the road structures and the use of the road construction. Pavement and foundation structures are treated as separate entities, which are combined when necessary.

Birgisdottir [8] developed the ROAD-RES model, a LCA tool for road construction and use of residues from waste incineration, the road system and the landfill being the core systems in the model. For the road system, the considered life cycle includes design, construction (earthworks, construction of pavement, additional work), operation (regular maintenance, pavement maintenance, winter service, leaching aspects) and demolition (removal of materials, area rehabilitation) stages.

Eco-Concrete, an Excel tool that applies three different LCA methodologies in accordance with the ISO standards to assess the impact of 10 fully-fledged functional units of concrete-based solutions on the environment, was developed in the frame of the EU Thematic Network “European Construction in Service of Society” [9]. The tool includes an assessment and comparison of the environmental impact of different pavement solutions.

3.2. Eco-labelling

The purpose of Eco-labelling is to provide information to consumers to enable them to select those products that are the least harmful to the environment, and to stimulate environmental concern in product development. The label itself has to be easy to understand and should provide very compact information.

According to the ISO 14020 standards, there are three types of eco-labels:

- type I - certified eco-labels, such as the *EU eco-label award scheme*, symbolised by the EU flower, define certain limits to achieve and independent verification is necessary;
- type II – self-declarations, usually focus on selected environmental aspects;
- type III – environmental product declarations (EPD), communicate the potential environmental impact of a product quantified by an LCA study.

3.3. Environmental indicators

ISO 14031 on Environmental Performance Evaluation elaborates the concept of environmental indicators. The principles for the derivation of environmental indicators are: comparability, target-orientated, balanced, continuity, frequency, comparability. Current research work aims to define and validate such indicators and to implement them to measure the sustainability of construction project. In the ECOserve project environmental indicators for innovative cement, aggregate, concrete products, and pavement were established. The selected indicators for pavement and their proposed units are:

1. transport of aggregates from quarry to production/construction site [t.km truck-eq per m^2 of pavement (whole life cycle), established for a 'reference' pavement location (e.g. national representative)];
2. CO_2 emission related to raw materials [t CO_2-eq per m^2 of pavement (whole life cycle)];
3. Life cycle costs [% relative to a reference pavement].

4. ECOCONCRETE LCA SOFTWARE TOOL

EcoConcrete is a tool to analyse the environmental performance of European ready mixed and precast concrete products. The software was developed under commission of the European concrete industry associated in a joint project group – members: BIBM (International Bureau for Precast Concrete), CEMBUREAU (European Cement Association), EFCA (European Federation of Concrete Admixtures Associations), ERMCO (European Ready Mixed Concrete Organisation) and EUROFER (European Confederation of Iron and Steel Industries).

EcoConcrete is a tailored software, peer reviewed, based on a MSExcel spreadsheet, which applies three different LCA methodologies in accordance with the ISO standards for the assessment of the impact of 10 fully-fledged functional units of concrete-based solutions on the environment (5 precast and 5 ready mixed products).

EcoConcrete is available under license.

The system requirements for using EcoConcrete are: Excel 97 or Excel 2000; Screen resolution: 1024 x 768 preferred; Pentium III processor preferred.

4.1. Structure of EcoConcrete

EcoConcrete contains two types of datasheets:

- Input datasheets:
  - Product - Select the product of interest;
  - Definition - Define the unit in which the results are expressed;
  - Composition - Define the concrete constituents and transport distances for delivery;
  - Life Cycle - Define the life cycle processes (such as maintenance, replacements and final waste scenario);

- Output datasheets:
  - Overview - Gives an overview of all filled out data related to the functional unit;
  - Main - Shows the environmental performances according to the CML 2000 methodology, the Eco-indicator 1999 methodology or the EDIP methodology.

The data input has the following structure:

1. Product: choice of the functional unit;
2. Functional Unit: description of amount, dimensions, service life;
3. Composition: type and amount of constituents (cement, aggregates, fillers, admixtures, water) and their transport (distance, mode);
4. Life Cycle: production, transport, construction, maintenance, demolition, recycling and landfill.

EcoConcrete uses life cycle impact assessment (LCIA) data as a basis (figure 3). This means that the data within EcoConcrete can never be disaggregated to life cycle inventory (LCI) or process data (right part of figure). EcoConcrete assembles an overall LCIA score for a product based on the separate materials and processes the user selects.

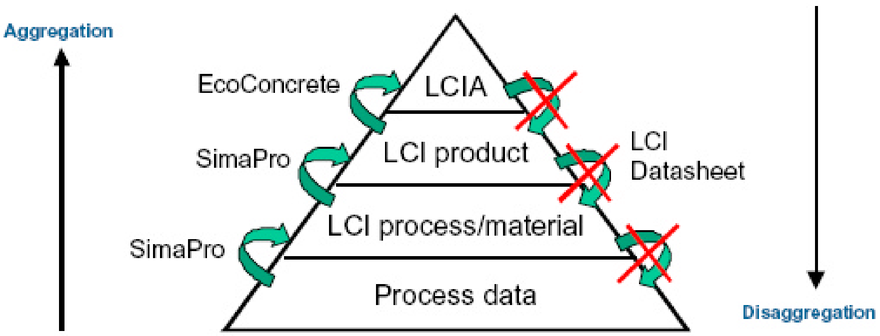

Figure 3. Level of aggregation of the LCIA data included in EcoConcrete

The LCIA data have been constructed with specialised LCA software (SimaPro). This is the input for EcoConcrete. The LCIA scores are based on site specific data for products and processes which are averaged and aggregated to arrive to data that can be regarded as representative for Europe.

4.2. Impact assessment methodologies

From different available impact assessment methodologies, EcoConcrete has chosen the following three methods:

- CML 2000 (Centre for environmental sciences, Leiden, NL) methodology;

- Eco-Indicator '99 methodology;
- EDIP (Environmental Design of Industrial Products) methodology.

All three methods:
- comply with the ISO 14042 standard;
- are used frequently;
- are operational on an European scale;
- are still under development. The scientific degrees of the various parts vary and are subject to ongoing improvements.

Differences occur in the number and type of environmental impacts (figure 4), the scientific background, the importance of environmental impacts in different nations, etc.

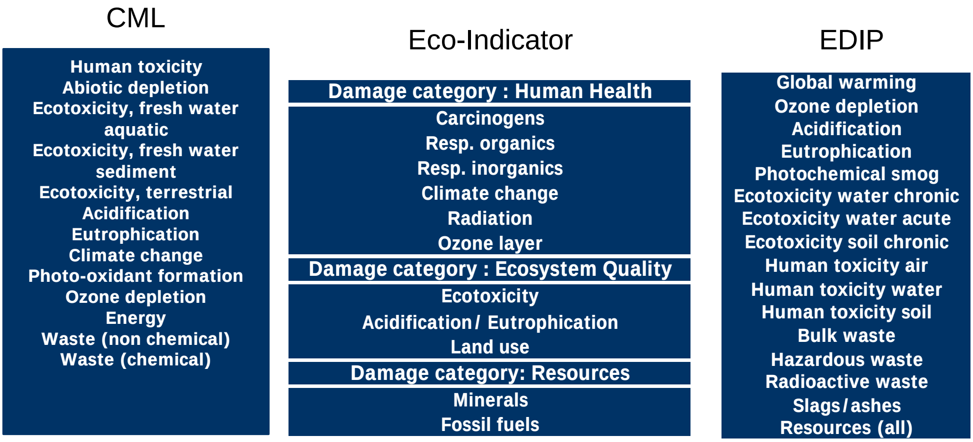

Figure 4. Impact categories included in the impact assessment methodologies

4.3. Results

EcoConcrete presents the LCIA results as tables and as graphs.

Graphs represent relative contributions per environmental theme. These themes vary along with the selected methodology. Tables represent absolute values per environmental theme. Absolute values are available for different materials and life-cycle phases.

An example of using EcoConcrete tool (input data, graph and table results) is presented in figure 5. The input data are based on information collected from the works carried out at the ALT-LIRA testing facility in Iasi in the frame of the FP6 EcoLanes project.

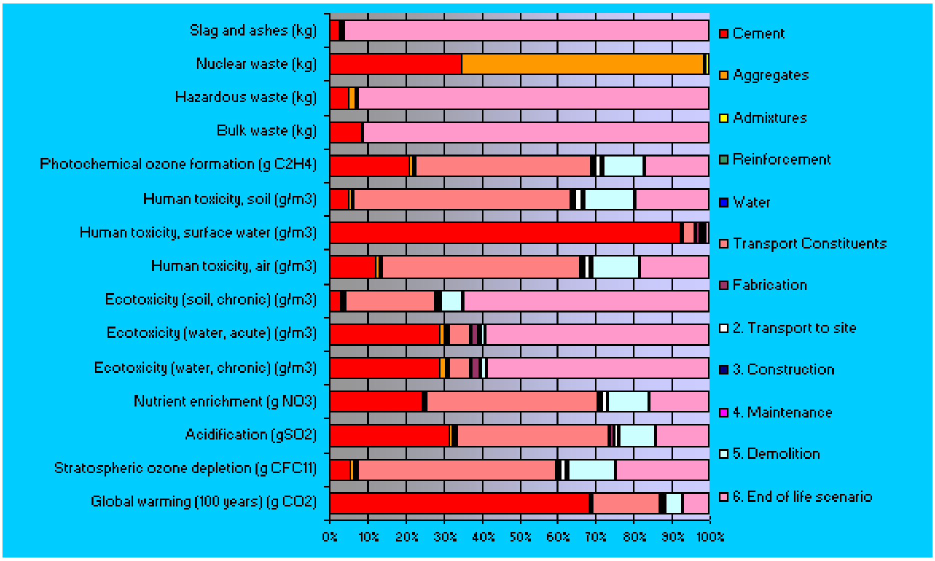

Figure 5. Graph results using EcoConcrete method

Results are given in terms of impact categories, based on both public and confidential inventories (given by the associations involved). It is impossible to disaggregate the results to see the inventories.

EcoConcrete does not provide:

- comparison with other materials (data base = only concrete);
- individual or national data (data base = European averages).

Environment should not be the only parameter a decision should be based on. EcoConcrete says nothing about the economic, social and other characteristics.

5. CONCLUSIONS

The construction of pavements requires a significant amount of non-renewable materials and energy. The sheer size of the energy and material investments dictates that a better understanding of the environmental and economic aspects of the use of virgin versus recycled materials might lead to a more sustainable future for the pavements construction sector.

It is of increasing concern that all emissions from transportation vehicles and the disruption of habitats and natural processes, caused by the extensive transportation infrastructure system and its use, are leading to gradual declines in biological diversity and ecosystem functions on regional and national scales. Climate change is also likely to affect ecosystem diversity and stability.

Environmental research related to transport pavements is a relatively new discipline and the data available in many cases are fragmented and contradictory. Information about environmental impact, particularly in the field of energy consumption, has to be further collected both from similar studies and from the construction companies.

Acknowledgements

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Diagnosis of cracking in concrete bridges structures

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Summary

A large percentage of the bridge structures in the nation's highway network are constructed out of reinforced concrete or prestressed concrete. Therefore it is important to have a very good knowledge of the basic characteristics of concrete in order to efficiently inspect bridge components build of this material.

The behavior of a bridge under traffic loads is strongly influenced by the properties of the materials used for the bridge. Therefore, the properties of construction materials are of great importance to the bridge inspector. Both physical properties, related to the intrinsic nature of the material and mechanical properties describing the structural behavior of the material are important to know these various strengths and weaknesses in order to understand the structural behavior of the entire bridge, as well as its many elements.

Assessing the integrity and safety of a bridge is possible if the various types of deterioration which can reduce the bridge's strength are understood.

KEYWORDS: concrete, cracking, classification, bridge

1. INTRODUCTION

Strength - plain, unreinforced concrete has a compressive strength ranging from about 175 MPa to about 415 MPa; however, its tensile strength is only about 10% of its compressive strength, its shear strength is about 12% to 13% of its compressive strength, and its flexural strength is about 14% of its compressive strength. Modulus of elasticity varies as the square root of compressive strength. In addition to elastic deformation, concrete exhibits long-term, irreversible, continuing deformation under application of a sustained load ranging from 100% to 200% of initial elastic deformation, depending on time.

Five principal factors that increase concrete strength are:
• Increased cement content
• Sound aggregates
• Decreased water to cement ratio
• Decreased entrapped air
• Increased curing time (extent of hydration)

Concrete is commonly used in bridge applications due to its compressive strength properties. However, in order to supplement the limited tensile strength of concrete, tensile steel reinforcement is generally used.

Steel reinforcement has approximately 100 times the tensile strength of concrete. Therefore, in reinforced concrete structures, the concrete resists the compressive forces and the steel reinforcement resists the tensile forces. Steel reinforcement for reinforced concrete is often referred to as "mild steel". The steel reinforcement is located close to the tension face.

Reinforcing bars are also perpendicular to the primary tension steel to resist stresses resulting from temperature changes and volumetric changes of concrete. This steel is referred to as temperature and shrinkage steel.

Steel reinforcing bars can be "plain" or smooth surfaced, or they can be "deformed" with a raised gripping pattern protruding from the surface of the bar. The gripping pattern improves the bond with the surrounding concrete. Modern reinforced concrete bridges are generally constructed with "deformed" reinforcing steel.

2. CAUSES OF CONCRETE CRACKING AND TYPES OF CRACKS

Concrete is by nature a brittle material, so reinforced concrete structures are destined to suffer cracking. Concrete is a mixture of cement, fillers and water. The cement, usually Portland cement, acts as a binder. Fillers are sand, gravel or other aggregate that make up the bulk of the concrete structure. Water reacts with the cement, causing it to harden. The ratios of the components depend upon the requirements for the particular concrete construction. Fillers are about 60 to 75 percent of the concrete, cement 10 to 15 percent and water 15 to 20 percent. Generally, less water in the mix makes stronger concrete, but it also makes the concrete mix harder to work with.

Cracking cannot be prevented completely using present techniques. Not all types of concrete cracking, however, pose problems; some are detrimental to structures but others are not. Damaging cracking induces those types that cause water leakage due to cracking throughout the element, excessive deflection, aesthetic concerns and damages to the durability of the structure.

Concrete is remarkably stable under even pressure loads, but concrete lacks tensile strength. If it is placed a sufficiently uneven load on a concrete structure it will crack. Particularly, thin or long concrete structures like slabs, beams or columns bear uneven stresses, so they are commonly reinforced with steel bars or fiberglass meshing.

Cracking can be categorized by phenomenon and causes as follows:
• cracking after reinforcement corrosion owing to the increase of corrosion of reinforcement;
• cracking before reinforcement corrosion that induces the corrosion of reinforcement;
• cracking representing the deterioration of concrete.
There are many common defects that occur on concrete bridges:

- cracking;
- scaling;
- delamination;
- spalling;
- chloride contamination;
- honeycombs;
- pop-outs;
- wear;
- collision damage;
- abrasion;
- overload damage;
- reinforcing steel corrosion;
- prestressed concrete deterioration.

A crack is a linear fracture in concrete. Cracks may extend partially or completely through the concrete member. Cracks occur in most civil engineering structures. Unexpected cracking of concrete is a frequent cause of complaints. Cracking can be the result of one or a combination of factors, such as drying shrinkage, thermal contraction, restraint (external or internal) to shortening, subgrade settlement, and applied loads. Concrete cracks are inevitable. Cracks are an inherent characteristic of concrete, and almost all concrete eventually develops cracks. In some cases, cracks do not harm the structural integrity of concrete. Other times, cracks can cause catastrophic failure of bridges. Regardless of the potential consequences of concrete cracks, they all arise from common causes. Understanding the causes helps prevent costly and dangerous flaws in concrete constructions of all sorts. Concrete is not a ductile material, it doesn't stretch or bend without breaking. That's both its greatest strength and greatest weakness. Its hardness and high compressive strength is why it used so much of it in construction. But concrete moves, it shrinks, it expands, and different parts of a bridge move in different ways.

As it moves, if it is tied to another element in the structure or even to itself, called restraint, which causes tensile forces and invariably leads to cracking. Restraint simply means that the concrete element (whether it's a slab or a wall or a foundation) is not being allowed to freely shrink as it dries or to expand and contract with temperature changes or to settle a bit into the subgrade.

Cracking can be significantly reduced when the causes are taken into account and preventative steps are utilized.

Causes of the cracks in generally are:
• cracking due to material problems;
• abnormal cracking due the structural problems;
• cracking due to shrinkage.

The major physical property of concrete that can lead to cracking is thermal expansion - concrete expands as temperature increases and contracts as temperature decreases.

Temperature rise (especially significant in mass concrete) results from the heat of hydration of cementitious materials. Hydration of cement is an exothermic process meaning it generates heat. As the concrete cools it contracts and in extreme conditions may contract in three days as much due to cooling as it could in a year due to drying conditions.

As the interior concrete increases in temperature and expands, the surface concrete may be cooling and contracting. This causes tensile stresses that may result in thermal cracks at the surface if the differential temperature between the surface and center is too great. The width and depth of cracks depends upon the differential temperature, physical properties of the concrete and the reinforcing steel.

After concrete is poured, the concrete increases in strength very quickly for a period of 3-7 days. Concrete cured for 7 days is about 50% stronger than uncured concrete. Ideally, concrete elements could be water cured for 7 days.
• porosity - because of entrapped air, the cement paste never completely fills the spaces between the aggregate particles, permitting absorption of water and the passage of water under pressure;
• volume changes due to moisture - concrete expands with an increase in moisture and contracts with a decrease in moisture.

On reinforced concrete, cracking will usually be large enough to be seen by the naked eye. However, on prestressed concrete, a crack gauge is the proper instrument needed to measure and differentiate cracks. Rust and efflorescence stains often appear at cracks. Both large and small cracks in main elements, especially in prestressed elements, should be carefully recorded.

3. TYPES OF CRACKS

There are several types of cracks depending on the chosen classification criteria:

3.1. The width

Cracks can be classified as hairline, medium, or wide cracks. Hairline cracks are usually cracks that cannot be measured with normal equipment. On conventionally reinforced structures, these hairline cracks are usually insignificant to the structural capacity of the structure. Medium and wide cracks are cracks that can be measured by simple means. These cracks can be very significant and should be monitored and recorded in the inspection notes. On prestressed structures, all cracks are significant. When reporting cracks, the length, width, location, and orientation (horizontal, vertical, or diagonal) should be noted. The presence of rust stains or efflorescence or evidence of differential movement on either side of the crack should be indicated.

3.2. The depth, surface and developing direction

Craze cracks are fine, random cracks or fissures in a surface of concrete. Crazing is a pattern of fine cracks that do not penetrate much below the surface and are usually a cosmetic problem only. They are barely visible, except when the concrete is drying after the surface has been wet.

Figure 1. Craze cracks (Source: http://www.concreteconstruction.net)

D-cracking represent a series of cracks in concrete near and roughly parallel to joints, edges, and structural cracks. D-cracking is a form of freeze-thaw deterioration that has been observed in some bridge elements after three or more years of service. Due to the natural accumulation of water in some exposed elements, the aggregate may eventually become saturated. Then with freezing and thawing cycles, cracking of the concrete starts in the saturated aggregate at the bottom of the element and progresses upward until it reaches the wearing surface. D-cracking usually starts near edges or joints of elements.

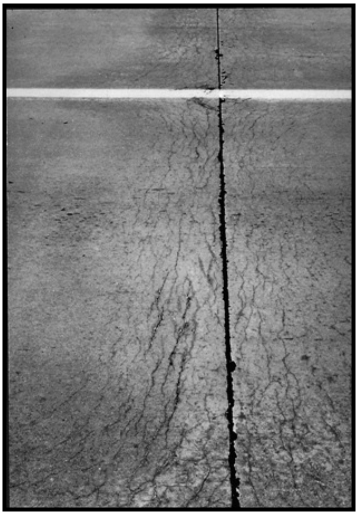

Figure 2. D-cracking (Source: http://www.concrete.org)

Pattern cracking are fine openings on concrete surfaces in the form of a pattern and results from a decrease in volume of the material near the surface, an increase in volume of the material below the surface, or both.

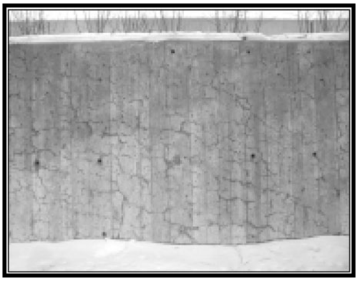

Figure 3. Pattern cracking (Source: http://irc.nrc-cnrc.gc.ca)

3.3. The cause that leaded to cracking:

a) *Plastic Shrinkage Cracking*

Contrary to popular terminology, concrete does not dry. Rather, it hardens, or cures. It is during curing that concrete becomes strong. The chemical process, called hydration, is a set of reactions that occur when water molecules bond with calcium trisilicate and calcium disilicate in cement. The hydrated compounds form a dense crystalline structure that binds the sand and aggregate into a single solid form. Other components in cement, tricalcium aluminate, tetracalcium aluminoferrite and gypsum – take part but are less important in the curing process. Plastic shrinkage cracking is the shrinkage that occurs in the surface of fresh concrete within the first few hours after it has been placed. Plastic shrinkage occurs as fresh concrete loses its moisture after placement but before any strength development has occurred. This type of shrinkage is affected by environmental effects of temperature (concrete and ambient), wind and relative humidity. It is a particular problem in hot weather concreting. This is an early age crack. While concrete is still plastic and before it has attained any significant strength. All concrete undergoes volumetric changes after placement. This volume change is caused by the loss of moisture as the concrete begins to dry. Approximately 80% of all water loss will occur within the first 24 hours. As the concrete hardens, due to restraint, it is unable to transfer the tensile stresses. When the tensile stress is greater than the tensile strength - first microscopic cracks, then large cracks will appear. Other factors that affect shrinkage cracking are: weather conditions, such as humidity, ambient temperature and wind velocity, a 40% drop in relative humidity can increase the evaporation rate by five times. Concrete temperature, mix proportions and aggregate type will also have an influence on the shrinkage.

Figure 4. Plastic shrinkage cracking (Source: http://www.cadman.com)

When water evaporates from the surface of freshly placed concrete faster than it is replaced by bleed water, the surface concrete shrinks. Due to the restraint provided by the concrete below the drying surface layer, tensile stresses develop in the weak, stiffening plastic concrete, resulting in shallow cracks of varying depth. These cracks are often fairly wide at the surface.

Actually, there are no specific admixtures developed to handle this type of cracking.

b) Drying Shrinkage

Because almost all concrete is mixed with more water than is needed to hydrate the cement, much of the remaining water evaporates, causing the concrete to shrink. Restraint to shrinkage, provided by the base, reinforcement, or another part of the structure, causes tensile stresses to develop in the hardened concrete. Restraint to drying shrinkage is the most common cause of concrete cracking. In many applications, drying shrinkage cracking is inevitable. Therefore, contraction joints are placed in concrete to predetermine the location of drying shrinkage cracks.

Since water is necessary for the hydration reaction of curing, evaporation on the surface of the concrete may cause the surface to cure more slowly than the interior. This uneven curing may lead to cracking. Low humidity and the heat released during the hydration reaction contribute to surface evaporation. This is why new concrete structures are commonly dowsed with water for several hours after the concrete is placed.

Figure 5. Drying shrinkage cracking (Source: http://www.members.optusnet.com.au)

Control joints are planned for cracks which allow movements caused by temperatures changes and drying shrinkage. In other words, if the concrete does crack, it is be better to have an active role in deciding where it will crack and that it will crack in a straight line instead of randomly. Joints in concrete can serve both to prevent cracking and as a decorative element.

c) Corrosion

Corrosion of reinforcing steel and other embedded metals is one of the leading causes of deterioration of concrete. When steel corrodes, the resulting rust occupies a greater volume than steel. The expansion creates tensile stresses in the concrete, which can eventually cause cracking and spalling.

Figure 6. Corrosion crack (Source: http://www.upload.wikimedia.org)

d) Alkali-aggregate reactivity

Alkali-aggregate reactivity is a type of concrete deterioration that occurs when the active mineral constituents of some aggregates react with the alkali hydroxides in the concrete. Alkali-aggregate reactivity occurs in two forms: alkali-silica reaction and alkali-carbonate reaction.

Indications of the presence of alkali-aggregate reactivity may be a network of cracks, closed or spalling joints, or displacement of different portions of a structure.

Figure 7. Alkali-aggregate reactivity cracks (Source: http://www.cement.org)

4. CRACKING DUE TO INTERNAL DEGRADATIONS OF CONCRETE

4.1. Honeycomb cracking

Honeycomb cracking is a condition of irregular voids caused by failure of the mortar to effectively fill the spaces between coarse aggregate particles. It may arise from congested reinforcement, insufficient cement content, improper sand-aggregate ratio, or inadequate placement techniques.

This kind of cracks is frequent to the top of works that has internal degradation of the concrete, being random and having an interrupted, intermittent path. It may take the shape of honeycomb cracking, with elements of 10 to 50 mm length and a depth of some centimeters. The width is variable upon the advancing degree of the reaction but remains of some 1/10 of a millimeter.

This type of cracking is sometimes underlined by the humidity. The contraction given by the drying is susceptible to provide the same type of cracking. Honeycomb cracking may be reduced with better vibration and improvement of workability.

Figure 8. Honeycomb cracking (Source: Handbook of concrete bridge management De Jorge de Brito)

4.2. Network cracking

Network cracking may appear to elements of bigger dimensions (10-40 cm) than of the honeycomb cracking. This type of cracking is observed especially in cases of alkali reactions. The width of cracks increases with the advance of the reaction. It can reach some millimeters in the case of a network with big elements. Like the width, the depth of cracks is developing and may have more than 10 cm.

Big elements may be crossed by smaller elements affected by network cracking, thus having a very complicated network and different dimensions. The cleaning of the top of works with water facilitates the cracks to be seen. The contractions due to drying (if the observed network is still formed by fine cracks), sulfated attack or freeze-thaw are susceptible to give the same type of cracking.

Figure 9. Network cracking (Source: http://www.filer.case.edu)

4.3. Oriented cracking

a) Cracking with one direction

When compression efforts opposes to the internal swelling the cracks are oriented preferentially on the direction of these efforts. It is the case of columns where vertical cracks may be observed. These vertical cracks may have also as origin an excess of compression or the corrosion of the vertical rebars with an inadequate concrete cover.

This is also the case of prestressed structures where the widening has a non prestressed direction. In the case of prestressed beams bridges the cracking is oriented perpendicularly on the longitudinal axis of the prestressing cables. These types of cracks appear at the building time, in the moment of tensioning the cables or during exploitation due to the corrosion of the prestressing duct and of the cables.

In some cases the superposing of the efforts given by an internal swelling reaction and the resulting efforts of classic actions applied on a structure leads to cracking with a preferential direction is identical with a mechanical one.

The mechanical origin cracking given by insufficient dimensioning of the elements may lead to confusion with the ones given by repeated loads applying cracking.

b) Two directions cracking

This cracking is due to internal swelling and alkali or sulfated reactions and appears on the direction of rebars on the surface layer.

The reinforcement is responsible for the normal shrinkage cracking. In the case of internal swelling all happens as the rebar is the start for cracking.

Conclusions on the cracking due to internal degradations of the concrete

In several cases of cracking through alkali reactions, particularly in the case of not reinforced elements of light reinforced, the depth of cracks is so that cracks cross each other. The element is presented as assemble of concrete blocks. The cracking favors the water penetration and aggressive agents. This water ‘fuels’ the internal swelling reactions but it provokes also the corrosion of the rebars, as shows the rusty colors on the exposed faces of the concrete elements.

The apparition and evolution of the cracking depends of the reaction’s kinetics. The time of the cracking depends especially on the period after the curing. Generally, deteriorations appear few years after the construction (2-5 years) and, rarely, 20-30 years after the construction.

Regarding the evolution of the cracking, the speed of widening of the main cracks may vary between 0,05 and 0,5 mm a year.

Cracking, like other deteriorations does not appear uniformly on the ensemble of a work. There is some heterogeneity in the area of deteriorations, function of several parameters as the reactive area extent, or the quantity of alkali in a certain area, the presence of water of humidity, the quantity and disposal of passive and active rebars, etc.

5. BRIDGE ELEMENTS AND ITS SPECIFIC CRACKS

5.1. Superstructure

a) Beams

On concrete beams, the two basic types of cracks are structural cracks and nonstructural cracks.

Structural cracks are caused by dead load and live load stresses and are divided into two categories:

The principal cracks of the reinforced concrete beams are the following:
- flexural cracks
- shear cracks
- both flexural and shear cracks

The presence of vertical cracks in the center of a vault or an arch or above the supports of a continuous beam can indicate a failure due to bending.

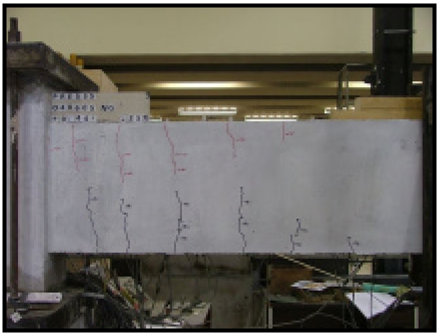

Figure 10. Flexural cracks (Source: http://www.methvin.org)

These vertical cracks develop from the extreme tensioned fiber of the concrete, meaning that it is starting from the bottom of the beam to the center of the vaults or arches or starting from the top of the beam above the supports of a continuous beam. They generally make the contour of the beam and attenuate while approaching its middle height. Their importance and their number decreases with the distance from the critical sections. Flexure cracks are vertical and start in the maximum tension zone or the maximum moment region and proceed toward the compression zone. At the mid-span of members, flexure cracks can sometimes be found at the bottom of the member where bending or flexure stress is greatest. Also look for flexure cracks at the top of continuous members near the piers.

The shear cracks are diagonal (approximately 45°) and are situated mainly close to the bearings. These cracks generally make the contour of the beam and attenuate with the closing up to its middle height. Their importance and their number decreases with the distance from the bearings. These cracks can generate three types of ruptures: the rupture by horizontal compression, the rupture by horizontal tension and the rupture by crushing or buckling oblique of web. The rupture by horizontal compression is characterized by a rotation of the beam compared to the top of the most critical crack and producing crushing of the concrete. The rupture by horizontal tension is due to insufficiency of anchoring of longitudinal reinforcements crossing the cracks. The rupture by crushing or oblique buckling of web occurs especially on beams with thin web. Shear cracks are diagonal cracks that usually occur in the web of a member. Normally, these cracks are found near the bearing area and begin at the bottom of the member and extend diagonally upward toward the center of the member.

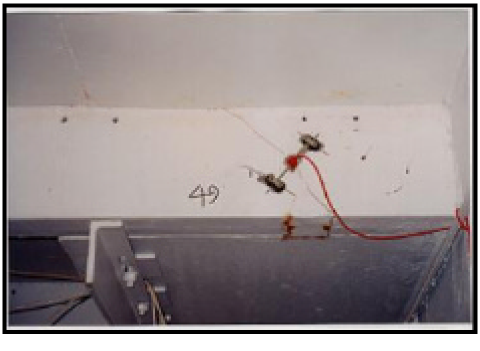

Figure 11. Shear cracks (Source: http://www.peterlindsell.co.uk)

Very rare torsion cracks may appear to the beams and are due to design or execution errors.

Torsion cracks generally propagate in a member making 45 degrees with a member's longitudinal axis. If torsional reinforcement is not provided, the initiation of torsion cracks means the failure of a reinforced concrete member.

When torsional reinforcement is provided in a member, the increase in applied torsion after the torsion cracking can be resisted depending on the amount of reinforcement. A number of torsional cracks can be observed. However, the tangential stiffness is gradually decreasing with the increase in the torsion angle. Finally, due to the crushing of concrete, a member will lose its strength.

Figure 12. Torsion cracks (Source: http://www.damninteresting.com)

Nonstructural cracks of the beams are divided into three categories:

- Temperature cracks
- Shrinkage cracks
- Mass concrete cracks

These cracks are relatively minor and generally do not affect the load-carrying capacity of the member. They can, however, provide openings for water and contaminants which can lead to serious problems.

Temperature cracks are caused by the thermal expansion and contraction of the concrete.

Figure 13. Temperature cracks (Source: http://www.irc.nrc-cnrc.gc.ca)

Shrinkage cracks are due to the contraction of concrete caused by the curing process.

Mass concrete cracks occur due to thermal gradients (differences between interior and exterior) in massive sections immediately after placement and for a period of time thereafter.

Figure 14. Shrinkage cracks (Source: http://www.inspect-ny.com)

b) Slabs

The principal cracks of the slabs are the following:

- transverse cracks
- longitudinal cracks

The transverse cracks generally develop in the intrados in the center of the span or above the supports in the extrados for a continuous slab. These cracks are structural and indicate a weakness to flexion of the slab.

The longitudinal cracks are due to the heat gradient, especially for the slabs of big width and including several continuous spans. Longitudinal cracks can also appear between the bearing supports, due the lack of braces in this area. In the angles of the very oblique bridges, a network of cracking perpendicular to the line of support may develop. The dissociation can result from the corner of the slab when the reinforcements are insufficient and that the directions of the longitudinal and transverse reinforcements are such that they form between them an acute angle.

c) Frames

The principal cracks of the frames are the following:

- transverse and horizontal cracks
- longitudinal and vertical cracks
- oblique cracks

Transverse and horizontal cracks can develop to the intrados of the frame in its center or to the extrados, to the junction of the intrados and the side walls and to the brackets if there's space for it. They can also be present in the side walls, generally to its middle height. These cracks are structural.

The longitudinal and vertical cracks are due to the heat gradient, especially for the frames of big width and including several continuous spans. The vertical cracks develop primarily in the side walls and in certain cases are prolonged in the brackets and may become longitudinal cracks in the intrados. These cracks are generally nonstructural.

Oblique cracks develop in the angles of the intrados and appear to the oblique bridges.

d) Arches

The principal cracks of the arches are the following:
- transverse cracks
- longitudinal cracks

Transverse cracks of concrete arches generally occur on the key or to the births. The transverse cracks on the key are due to differential settlement of supports. The transverse cracks to the births are specifically to the arches with deck on top or in the middle. Their positions are on the intrados of the arch to the births side and are active under thermal effect. The cause is given by the deformations of the arch that are limited by the births transmitting important efforts.

Longitudinal cracks of the concrete arches generally occur to the tympanum or on the arch. The longitudinal cracks on the tympanum are due to differential settlement of bearings or a differential settlement on a same bearings line. The longitudinal cracks on an arch are due to the insufficiency, even an absence, of the transverse reinforcement. The concrete is not enough disposed to face the tractions resulting from the compressive forces in the arch. Such cracks occur also when the arches are made up of box girders, because of different rigidities of the web and the frame.

e) Cross Beams

This vertical or tilted cracking appears to the connections between the principal beams and the cross beams. This cracking corresponds primarily to the phenomena of differential shrinking of the concrete or, eventually, to the setting in tension of the prestressing reinforcement provoking dissymmetrical deformations between beams, soliciting abnormally the cross beam. In the case of the bearing cross beams, a cracking of the cross beam is often revealing its insufficient resistance, following differential settlements of the bearings, or a too big solicitation of the cross beams under the life loads.

f) Other Decks

In concrete bridge decks, temperature and shrinkage cracks can occur in both the transverse and longitudinal directions.

Prestressed concrete sections can lose their strength through several forms of concrete deterioration. Prestressed concrete members are especially sensitive to corrosion and fatigue in isolated cracks. The corrosion of prestressing wire can lead to a failure of the member. Loss of bond between the prestressing steel and the concrete can result in member failure. Unbonded members are subject to zipper effects. Relaxation of prestressing steel due to high, sustained tensile stress can cause a gradual decrease in strength over time. Shrinkage of the concrete causes a further relaxation in the prestressing steel, thereby lowering the strength of the member. In addition, creep in the concrete will cause the member to shorten, causing further relaxation in the steel tendons, which results in additional strength loss.

5.2. Substructure

In retaining walls and abutments, temperature and shrinkage cracks are usually vertical, and in concrete beams, these cracks occur vertically or transversely on the member. However, since temperature and shrinkage stresses exist in all directions, the cracks could have other orientations.

The vertical cracks that develop in the sole and the front wall of the abutments may be caused by:

- non uniform shrinking of the concrete
- errors in design
- differential settlement.

The exposed surface of the concrete front wall of the abutment is drying while the surface which is in contact with the embankment remains humid. Under these conditions, the contraction can cause tension cracks in the wall.

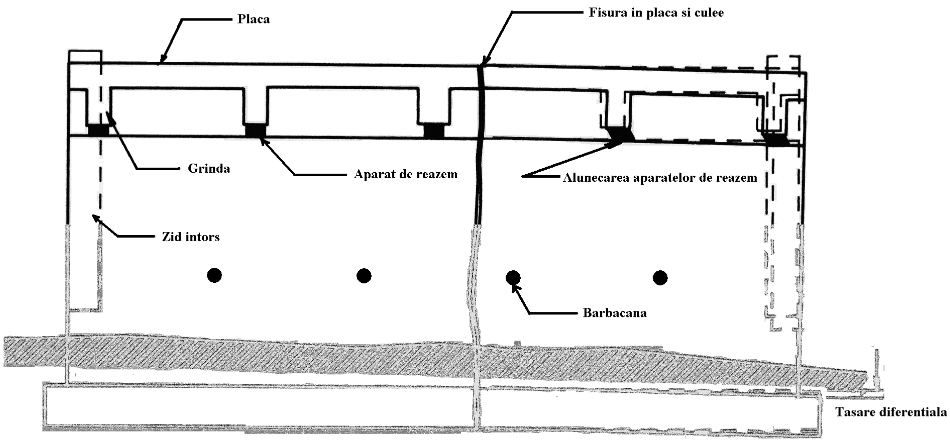

Figure 15. Differential settlement cracks

In the design of the abutments, one assumes that the retaining walls have no influence on the abutment, but in reality they act like buttresses. In this respect, the front wall of the abutment may be solicited to longitudinal flexion between the buttresses, which requires horizontal reinforcements in the front face of the wall. In the older abutments, the horizontal reinforcements are often concentrates at the back of the walls; it is possible that vertical cracks develop on the front face of these abutments. The wing walls and the retaining walls are also designed like they are independent of the abutment, but they are always connected. The least compressing of the wing walls and the retaining walls causes traction efforts not envisaged and cracks on the junction with the front wall of the abutment. The differential settlement which occurs in the longitudinal direction of the abutment causes usually cracks in the guard-strike, the front wall and same in the sole of the abutment.

6. RESISTANCE AND DURABILITY AGAINST CRACKS

Reinforcing bars can be used to increase the compressive strength of a concrete member. When reinforcing bars are properly cast into a concrete member, the steel and concrete acting together provide a strong, durable construction material.

In addition to reinforced concrete, prestressed concrete, using high strength steel wires, can be used in bridge applications. To reduce the tensile forces in a concrete member, internal compressive forces are induced through prestressing steel tendons or wires. When loads are applied to the member, any tensile forces developed are counterbalanced by the internal compressive forces induced by the prestressing steel. By prestressing the concrete in this manner, the final tensile forces are typically within the tensile strength limits of plain concrete. Therefore, properly designed prestressed concrete members do not develop flexure cracks under service loads.

In pretensioned members, transfer of tendon tensile stress occurs through bonding, which is the secure interaction of the prestressing steel with the surrounding concrete.

In post tensioned members, transfer of tendon tensile stress is accomplished by mechanical end anchorages and locking devices. If bonding is also desired, special ducts are used which are pressure injected with grout after the tendons are tensioned and locked off. This is accomplished by casting the concrete in direct contact with the prestressed steel. In post tensioned members, bonding is accomplished by injecting grout into the ducts after the high tensile steel is stressed.

For purposes of crack control in end sections of pretensioned members, the prestressing steel is sometimes unbonded. This is accomplished by providing a protective cover on the steel, preventing it from contacting the concrete. For post tensioned members, when bonding is not desirable, grouting of tendon ducts is not performed and corrosion protection in the form of galvanizing, greasing, or some other means must be provided.

Cracking in concrete can be reduced significantly or eliminated by observing the following practices:

1. Use proper subgrade preparation, including uniform support and proper subbase material at adequate moisture content.

2. Minimize the mix water content by maximizing the size and amount of coarse aggregate and use low-shrinkage aggregate

3. Use the lowest amount of mix water required for workability; do not permit overly wet consistencies.

4. Avoid calcium chloride admixtures.

5. Prevent rapid loss of surface moisture while the concrete is still plastic through use of spray-applied finishing aids or plastic sheets to avoid plastic-shrinkage cracks.

6. Provide contraction joints at reasonable intervals, 30 times the slab thickness.

7. Provide isolation joints to prevent restraint from adjoining elements of a structure.

8. Prevent extreme changes in temperature.

9. To minimize cracking on top of vapor barriers, use a 100 mm thick layer of slightly damp, compactible, drainable fill choked off with fine-grade material. If concrete must be placed directly on polyethylene sheet or other vapor barriers, use a mix with low water content.

10. Properly place, consolidate, finish, and cure the concrete.

11. Avoid using excessive amounts of cementitious materials.

12. Consider using a shrinkage-reducing admixture to reduce drying shrinkage, which may reduce shrinkage cracking.

13. Consider using synthetic fibers to help control plastic shrinkage cracks.

7. CRACK MONITORING

One method for cracks monitoring is the crack diagram on four directions.

Two perpendicular axes are drawn on the monitored element, the equal angles line and its equal segments. The widths of cracks on the four directions are drawn for each segment. The sum of crack widths divided by the length of each axis is the cracking index on each direction. These measurements must be repeated regularly to monitor the evolution in time.

Monitoring the widening evolution of some cracks may be realized using movement sensors.

From the point of view of mechanisms that leaded to the apparition of the crack in concrete we may have one of the following mechanisms:

a) Settlement of the soils supporting the concrete element

Loss of support beneath concrete structures, usually caused by settling or washout of soils and sub base materials, can cause a variety of problems in concrete structures, from cracking and performance problems to structural failure. Loss of support can also occur during construction due to inadequate formwork support or premature removal of forms. Settlement cracking takes place when the soils or fill beneath the element have not been adequately compacted to provide a consistent level of support for the element to limit the bending stresses which crack the concrete. Settlement can be controlled with consistent preparation (compaction) of the base supporting the element.

b) Restraint of horizontal movement due to fixed foundation elements

Elements placed against fixed foundation elements (frost foundations, light standards, etc.) produce cracks caused by bending forces as the element moves on the surface while the fixed foundation does not. This mechanism is controlled by placing isolation joint material between the element and the fixed foundation to allow the elements to move independently, thus limits the bending stresses and subsequent cracking.

c) Overloading, applying a load larger than the element was designed to support

Overload cracking is easily controlled with proper thickness design of the element considering the largest load that may be applied to its surface.

d) Environmental cracks

Tree roots can crack concrete.

Stresses on concrete don't come only from the intended use of the concrete. Freezing and thawing places stress on concrete. Plant roots infiltrate small fissures in concrete. Chemical exposure weakens the molecular structure of concrete. These are common environmental factors that also promote concrete cracking.


Figure 16. Environmental cracks (Source: http: //www. www.ehow.com;

http://www.imagination.lancaster.ac.uk)

When examining the deterioration conditions of a concrete member, access to the previous inspection report is desirable. This allows the inspector to note the progression of concrete deterioration and provides a more meaningful evaluation and inspection report.

The inspection of concrete should include both a visual examination and a physical examination.

One of the primary forms of deterioration observed during the visual examination is cracking. All cracking should be described and recorded. Future inspections will detect changes in the crack patterns or sizes which indicate active distress.

Monitoring the changes in crack width is an important diagnostic technique for determining the cause and specifying the remedial work. Crack depth gauge is to be used.

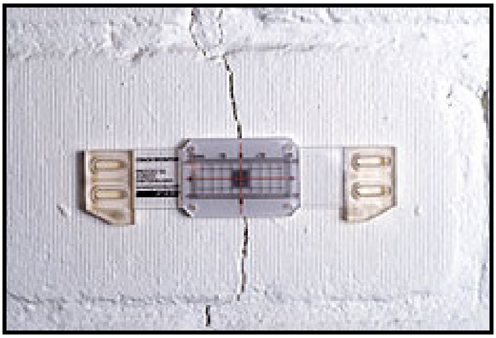

Figure 17. Crack depth gauge

8. REHABILITATION

Rehabilitation requires the selection and combination of appropriate methods according to the type and extent of damage. The deteriorated concrete must be removed. Representative rehabilitation is crack repair, improving the waterproofness and durability of cracked areas. Possible methods to be used are: surface treatment, injection, filling, waterproofing.

Cracking is caused by almost all of the factors that cause deterioration of concrete. An appropriate method should be selected according to the purpose (waterproofness or durability), state and cause of cracking, and crack width.

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Analysis of Costs Categories for a Bridge Service Life

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Summary

The present paper wishes to present the cost components which can be determined during a bridge service life, from conception to replacement.

This type of analysis of cost categories is efficient in bridge design and for the decisions management regarding the economic efficiency of construction, rehabilitation and bridge maintenance projects.

Considering the fact that the deteriorations which may appear in a bridge structure are quite different so can be the cost categories. The optimal allocations of cost categories should be in conformity with BLCCA – Bridge Life Cycle Cost Analysis.

Considering all these in the paper is being determined the technical evolution of a bridge condition during the service life and the financial component of this evolution.

KEYWORDS: cost components, service life, BLCCA – Bridge life cycle cost analysis, technical state.

1. INTRODUCTION

The bridge is considered a structure with a service life consisting of several years. By the end of this time period, the bridge can be replaced and can be taken the decision of a new structure construction.

The new structure follows the steps of any construction work, in total they compose a cycle which can be repeated over and over again. This is the life cycle of a bridge structure.

Evaluation of the value and effects of the costs during the service life, the way these costs appear and act is called "costs analysis during construction life cycle". This type of evaluation has become the main instrument for decision making in patrimonial management.

Life cycle costs represent the gathering of all costs estimated to appear from conception to replacement of a bridge structure. The objective of this analysis is to choose the most efficient of different possible projects, the one with optimal cost – efficiency report.

Each of the intervention works for bridges has a cost which depends on the bridge dimensions and degradation importance. The works execution is being related to the manager ability to mobilize the necessary financial resources. The incapacity to find the funds leads to works delay for the next budgetary years. The lack of works execution leads to an increase of degradations, of the defaults and finally to the increase of intervention works value.

Some states have recognized the need to see beyond the initial costs and have taken the practical steps for initiation and implementation of costs analysis in bridge management. The most important challenges which have to be faced in order to improve the process are the lack of complete and quality information. The costs estimation must be based on historical data regarding the contracts and activities but also the expert’s predictions.

The life cycle costs analysis sustains the bridge design and decision management in order to help the engineers to evaluate the economic efficiency of the projects proposed for construction and rehabilitation. All the costs of the agencies involved in each alternative are considered in the analysis including costs for:

- Design,
- New construction,
- Unforeseen and administration costs,
- Inspection and maintenance,
- Repairing,
- Rehabilitation and consolidation,
- Surface enlargement, demolition and replacement,
- Superstructure demolition and replacement,
- Total demolition of the bridge and replacement etc.

The analysis should also include the indirect costs such as the users costs and vulnerability costs, in completion to the costs paid by the bridge owner’s agencies.

2. THE USE OF COSTS ANALYSIS IN BRIDGE MANAGEMENT

„The bridge management” refers to the planning, design, operation and maintenance activities which create a bridge configuration during its service life.

The new bridge management programs wish to impose the acceptance of more important expenses for sustainable elements during initial construction in order to reduce the future interventions frequency or adoption of a more expensive project in view of an easier and less expensive future maintenance.

Following the model of life cycle cost analysis for highway projects, has been realized the structure of a similar analysis for bridge projects. This type of analysis is being useful to the officials from bridge area interested to select an action for bridge design and improvement. The general idea is to minimize the costs during a bridge service life.

The Bridge Life Cycle Cost Analysis - BLCCA refers to costs evaluation and comparison for bridges.

This type of analysis reveals the fact that the expenses realized during a bridge service life aren't punctual. It's impossible to gather a sum which is used at a certain moment without other costs. If the sums initially invested for a bridge are big, the money necessary for future maintenance, repairing, rehabilitation and replacement represent a more important value.

Similar to any construction, the bridge has a service life during which the bridge passes different phases: design, construction, exploitation, maintenance.

All the previous elements are presented in figure 1 where is reflected the interventions effect of works on bridge condition, interventions which lead to the condition improvement.

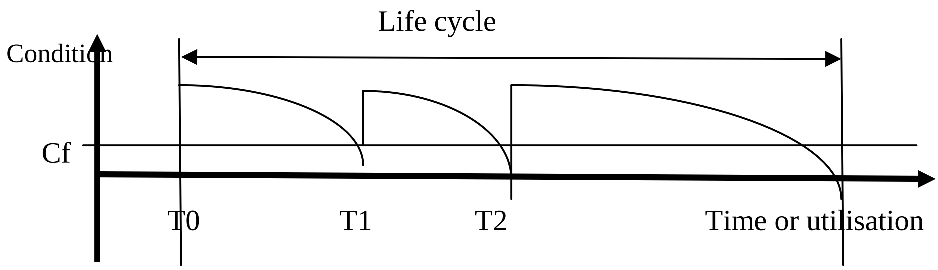

Fig. 1 Evolution of bridge technical state during service life in the case of repairing and rehabilitations

Figure 1 presents the extended evolution of repeated interventions during a bridge service life. T0 represents the moment the bridge is gave into exploitation. In the case of T1 and T2 are realized complex interventions on the bridge deteriorated elements. The interventions raise the bridge condition value.

3. COST CATEGORIES IN BLCCA ANALYSIS

The bridge costs appear as a consequence of the actions of a transportation agency in order to facilitate the highway and bridge utilization activities. These actions take place during a bridge service life and develop from initial development of a bridge to its replacement.

The costs discovered during BLCCA analysis can be classified in three categories: agency costs, users costs and vulnerability costs.

3.1 Agency costs for bridges

Design and regulations: include all the studies and other reviews, consultancy made before financial requests for a new bridge or a new major rehabilitation.

Acquisitions and other compensations: these costs include expenses for lands and compensations for discounts in lands value associated to reduced access, impact on environment or different restrictions imposed by the construction.

Construction: includes the administrative and contractual costs necessary for the development of the bridge and annexes. Here can be included the expenses due to the delays in projects execution.

Maintenance and repairing: refers to elements such as:

- periodical activities for bridge conditions maintenance called normal maintenance,
- actions for repairing or replacement of bridge elements which represent a threat for the users.

Contractual impulses and constraints: some agencies use impulses and constraints in contracts in order to encourage a faster finalization of the constructions and major rehabilitation and repairing projects, discourage the traffic obstruction and reduction in the time and severity of traffic disturbance.

These instruments take the form of penalties for every delay day towards the established date for contract finalization, taxes for contractor for every hour in which the traffic is blocked or extra payments for every day in which the contracts finalization precedes the contracts finalization date.

Demolition and remediation: these actions refer to bridges out of service and partial or total demolished.

Inspections: refer to programmed and special inspections due to deteriorations, extraordinary events or bridge particular problems.

Site and administrative services: refer to administrative activities of the agencies for quality assurance and verification of the payments made during construction, rehabilitation and repairing works.

Replacement and rehabilitation: the bridge replacement can include only the superstructure or total replacement of the foundation, infrastructure and superstructure.

3.2 Users cost

User's costs must result from, for example, observations regarding additional fuel consume and time lost value due to traffic gems.

Delays due to traffic gems: include delays imposed to users:

- by temporary close of the bridge for maintenance, repairing or rehabilitation,
- traffic gems when different bridge components closes are slowing the traffic and determine secondary delays,
- slowing the traffic due to road conditions,

Traffic deviations and diversions determined by delays: refer to costs imposed to users necessary for deviation towards other routes because a bridge can't adapt to the weight or the dimensions of a vehicle or due to bridge close or strong congestion.

Vehicle destruction: the bridge deck and work zones conditions can increase the possibility that the vehicles which cross over the bridge to be distressed, for example by the unmanaged pavement or accidental, obstructions and other. These costs can be estimated as a proportion in traffic level. These costs are being paid to users and included into agency costs.

Environment destruction: the users can stand costs of environment destruction associated to bridge management if the soil is being modified or destroyed, if are introduced in water pollutants or wastes and the air in adjacent areas is being polluted.

3.3 Vulnerability costs

The bridges are exposed to circumstances which imply the dangers manifested under the form of floods, seismic events or traffic events which can or not determine perturbations or destructions. The costs which should be sustained by agencies for destructions repairing determined by an event can be avoided if the bridge has been designed, constructed and maintained in conformity with current norms and standards.

The annual vulnerability cost can be determined for every potential danger.

Structural destructions regarding the loading

- Overloading,
- Fatigue,

Destructions determined by collisions

- Collisions in traffic,
- Of the vehicles with the structure (vertical, horizontal),
- Other collisions with the structure (train, ships etc)

Destructions due to earthquakes
Destructions determined by floods
Destructions connected to erosion
Obsolescence

Fig. 2 Components of vulnerability costs

Fig. 2 presents the types of dangers which can be considered as components to take into consideration in the bridge costs analysis: climate, geology, traffic and other local facts.

Structural deteriorations caused by loads applied to the construction: these costs include looses of the structural integrity due to overloading or excessive obsolescence in key structural components.

Destructions due to collisions: include the events in which over dimensional vehicle or out of control hit the bridge with enough force or determine fires or chemical materials overflow which threaten the structure integrity.

Destructions due to earthquakes: the bridges placed in areas with high seismic risk are susceptible of destructions, which has determined the adoption of more important design standards for moment and lateral force resistance.

Destructions connected to floods: are similar to the destructions due to earthquakes, with structure destructions due to lateral forces imposed by strong floods and impact of overflows with the superstructure and bridge support.

Destructions related to the erosion: the bridge pillars erosion, the foundation material erosion is a dynamic phenomenon related to the water depth and flux angle, pillars form and depth, soil characteristics etc.

Resources obsolescence: bridge obsolescence is determined by a number of factors including technological changes, regulatory changes, social and economic changes. An old bridge is not necessarily incapable to support the traffic or dysfunctional. It just doesn't rise to the current needs and expectations.

3. CONCLUSIONS

Internationally there's not a clear delimitation of costs categories necessary during a bridge service life. Unfortunately in the costs analysis only 10% of the transportation agencies take into consideration also the users costs. The majority considers only the agency costs.

International Organizations as main investors in transportation construction works impose the use of Life cycle cost analysis for the optimization of financial implication.

Regarding the connection of the costs with the bridge service life has come to the conclusion of the importance of bridge construction works monitoring and also to realize the interventions to the optimum moment, avoiding their delay.

Between the obstacles of costs analysis implementation for bridges can be considered the lack of complete and quality information. For a bridge it is very important to take into consideration the projected service life and follow a strict intervention on the structure at regular time intervals and by quality works.

The cost components which should be taken into account in all the economic analysis projects for bridges are the agency, users and vulnerability costs.

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EcoLanes: Paving the Future for Environmentally-Friendly and Economical Concrete Roads

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Abstract

With increasing oil prices the future of asphalt roads on deep foundations is becoming increasingly uncertain, due to increased costs as well as political and environmental concerns. Concrete pavement bases can reduce the foundation layers and decrease or eliminate the asphalt topping. Prior to the on-going oil crisis, concrete bases were generally more expensive to construct, and were mostly used in heavily trafficked sections and to reduce maintenance. Concrete bases are in general reinforced with steel mesh to improve their strength characteristics; however, this process is labour intensive and has health and safety problems. Premixed steel fibre reinforcement is also used to replace the mesh and provides a less laborious construction technique. Steel fibres are normally derived from virgin steel wire and are more expensive than mesh reinforcement. Recently, it has been shown that recycled steel fibres, produced from post-consumer tyres, offer an attractive low cost alternative solution. Recycled aggregates, pulverised fuel ash and low energy cements may also be employed to reduce costs and energy input. This paper presents an overview of the EcoLanes research project, which investigates the above issues and aims to develop long lasting rigid pavements for surface transport by utilising roller compaction techniques and low cost steel fibre reinforced concrete.

Keywords: RCC, concrete pavements, LLRP, recycled steel tyre-cord fibres

1. INTRODUCTION

A massive and targeted investment is currently required for the rehabilitation and extension of the European surface transport infrastructure, to provide a system able to respond to the needs of the enlarged European Union (EU), (ECTP, 2005).

The main element of surface transport infrastructure is the pavement, which can be either flexible or rigid. Flexible pavements are normally constructed with asphalt concrete, whereas Portland cement concrete is used for rigid pavements. The increasing demand to adopt innovative and durable construction practices has led to the wider use of concrete pavements, which in general have a longer working life than asphalt pavements (Embacher and Snyder, 2001).

Concrete pavements are normally reinforced with steel mesh reinforcement to improve their mechanical behaviour, reduce number of joints and minimise the foundation depth, required to achieve the necessary structural performance. Steel fibres mixed with wet concrete can be used to replace rebar reinforcement and thus reduce the labour costs associated with the placement of the reinforcement and speed-up the construction process. In general, the application of steel fibres is restrained by the high cost of the fibres, especially in countries where labour costs are relatively low. The price of industrially produced steel fibres, ranging from €800 to €15,000 per tonne, is at least 20% higher than the price of conventional steel bars and, since they are randomly mixed in a layer, larger volumes are needed to achieve equivalent structural performance. Steel fibres recycled from waste streams, such as post-consumer tyres (Figure 1), can offer an alternative solution to industrially produced steel fibres, since the value of recycled steel fibres (as scrap material) ranges from €20 to €150 per tonne (Pilakoutas *et al.*, 2004).

Figure 1 - Mechanical shedding of post-consumer tyres: recovery of rubber and steel

Concrete pavements are constructed using either a wet or a dry mix. The wet mix is placed and compacted with conventional concreting techniques which are laborious and require side formwork. Dry mixes can be placed with a modified asphalt paver and compacted by vibratory rollers (Figure 2) and, thus, provide a fast, cost-effective, and durable solution. In dry mixes, steel fibres are difficult to incorporate, but have been shown in laboratory experiments to improve the mechanical properties of dry roller-compacted concrete (Nanni, 1989).

Depending on the prevailing material and energy prices, concrete pavements could be made more cost-effective than asphalt pavements (Johnson, 2008). However, to provide a truly sustainable solution, it is necessary to reduce the energy consumption during the production of this type of pavement as well as the cost of materials. The main energy component of concrete pavements (from extraction of raw material through the placement of the pavement) is the energy used for the manufacture of cement and steel reinforcement (Zapata and Gambatese, 2004). Through the utilisation of low energy cements as well as recycled materials, such as steel tyre-cord fibres, pulverised fuel ash, and aggregates obtained from construction waste, it is possible to minimise the cost associated with the energy consumption of concrete pavements. In addition to the energy consumption, the use of low-cost recycled materials will further reduce the material cost of concrete pavements.

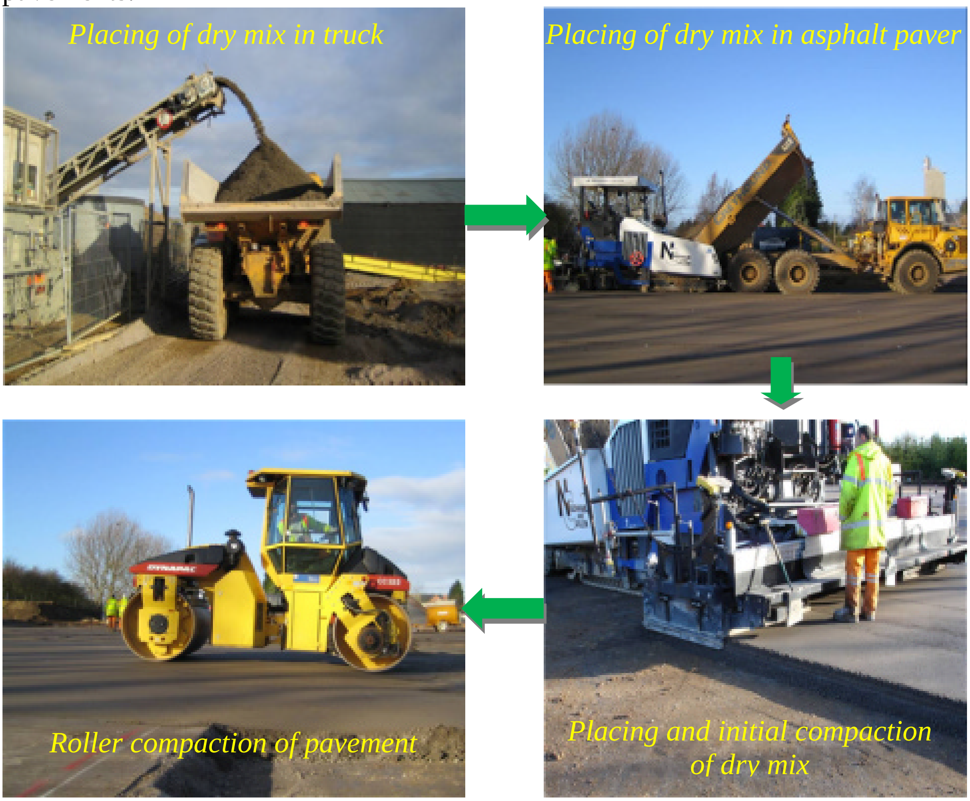


Figure 2 - Construction of roller-compacted concrete (RCC) pavements

An EC FP6 STREP project, called EcoLanes, is currently investigating the above issues and aims to develop long lasting pavement infrastructure for surface transport by using roller-compaction techniques and dry steel fibre-reinforced concrete (EcoLanes, 2008). This paper presents an overview of the EcoLanes, including its aims and objectives. In addition, the paper elaborates on the experimental work carried out to develop steel fibre reinforced roller-compacted concrete.

2. ECOLANES PROJECT OVERVIEW

EcoLanes is funded, for three years, under the priority thematic area of Sustainable Surface Transport of the 6th framework programme of the European Community. The project, which started in late 2006, draws expertise from six European countries and its consortium comprises four universities, three industrial partners, the European Tyre Recycling Association and three end-users (EcoLanes, 2008). In early 2008, Universidade Federal do Rio Grande do Sul was invited to join the EcoLanes consortium.

The main aim of this project is the development of pavement infrastructure for surface transport using roller compaction techniques (Figure 2), based on existing asphalt laying equipment, and dry concrete mixes reinforced with steel tyre-cord fibres. The benefits of the new construction concept will be manifold, such as to reduce construction costs by 10-20%, reduce construction time by at least 15%, reduce the energy consumption in road construction by 40%, minimise maintenance, use post-consumer materials in road construction and make tyre recycling more economically attractive. Through its nine work-packages (Figure 3), the project will deliver new processes, models for life-cycle assessment and costing, and design guidelines. The results of the project will be validated by constructing full-scale demonstration projects in four diverse European climates and economies (i.e. Cyprus, Romania, Turkey and United Kingdom), which will be constructed in early 2009.

To achieve its aims and objectives, the project has to overcome scientific and technological barriers in fibre processing, concrete manufacture and road design.

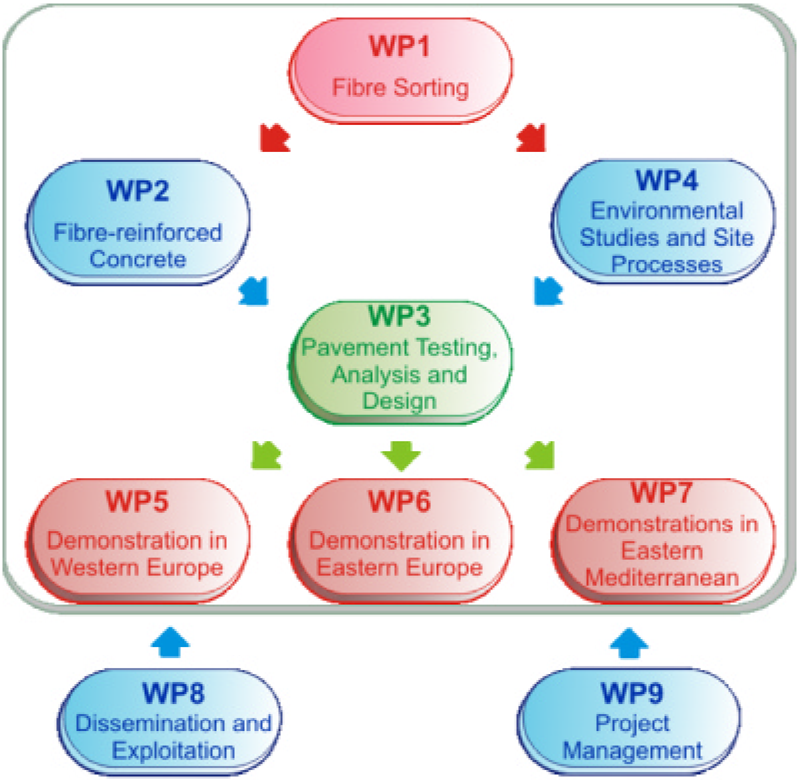


Figure 3 - EcoLanes work-package overview

2.1. Fibre Processing

Previous research has demonstrated that steel tyre-cord fibres improve the mechanical properties of concrete (Pilakoutas *et al*., 2004). However, one of the main problems, encountered when mixing steel tyre-cord fibres in fresh concrete, is the tendency of the fibres to ball together, which spoils the concrete (Pilakoutas *et al*., 2004). Fibre balling is mainly caused by the irregular geometry of the fibres (Figure 4). Thus, the EcoLanes project has already developed techniques and equipment that minimise the geometrical irregularities of the steel tyre-cord fibres and arrive at the optimal lengths required to best utilise the steel strength and avoid balling.

(a)

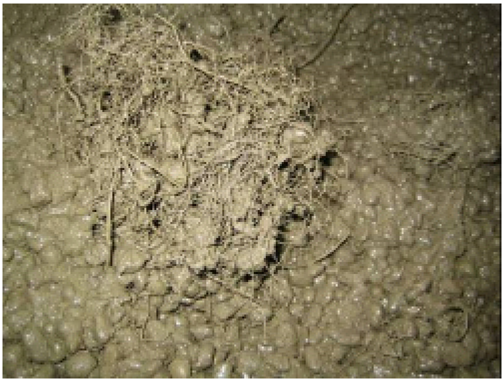

(b)

Figure 4 – (a) Irregular steel tyre-cord fibres, produced from the shredding of post-consumer tyres (b) Fibre balling in wet-mix concrete

2.2. Concrete Engineering

Despite the improved mechanical properties of steel fibre-reinforced roller-compacted concrete (SFR-RCC), the addition of steel fibres can lead to compaction problems and, thus, affect the concrete density. Damage may also be caused to the steel fibres during the compaction process. These problems are being tackled by EcoLanes though extensive laboratory experiments (section 3) prior to the full scale trials of the demonstration projects.

In addition, SFR-RCC has not been used extensively, up to now, due to the difficulties in incorporating the fibres in the dry mix. To eliminate this technological barrier, the EcoLanes consortium is examining various industrial processes and equipment which could be used to successfully disperse steel fibres in dry mixes, and maximise the amount of fibre content added to the mix without attaining balling. In June and September 2008, pre-demonstration trials were carried out in the United Kingdom and Romania, respectively, to assess the suitability of pan mixers for producing SFR-RCC. The results of the trial (Figures 5 and 6) were satisfactory as the fibres were evenly distributed in the dry mix with minimal fibre balling.

Figure 5 – SFR-RCC produced in an industrial pan mixer

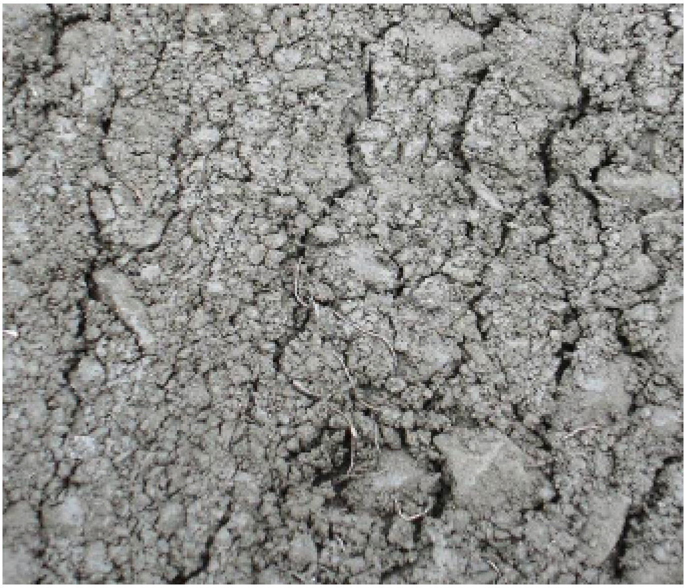


Figure 6 – Surface of SFR-RCC pavement following paver compaction

2.3. Road Design

The economic and sustainable design of concrete pavement is a complex calculation requiring input ranging from the material (physical, mechanical, chemical) characteristics and cost, energy inputs, cost of labour, equipment and fuel, to advanced numerical techniques. All these parameters will be determined through the development of the concept of the long lasting rigid pavement (LLRP), which will also consider the results of life cycle assessment and costing. The LLRP concept is currently being technically validated on a circular accelerated testing facility (where sections of selected SFR-RCC mixes will be subjected to 1.5 million load cycles), and numerical analyses and parametric studies will be carried out to develop design models for LLRPs (Andrei *et al.*, 2007; Vlad *et al.*, 2008; EcoLanes, 2008).

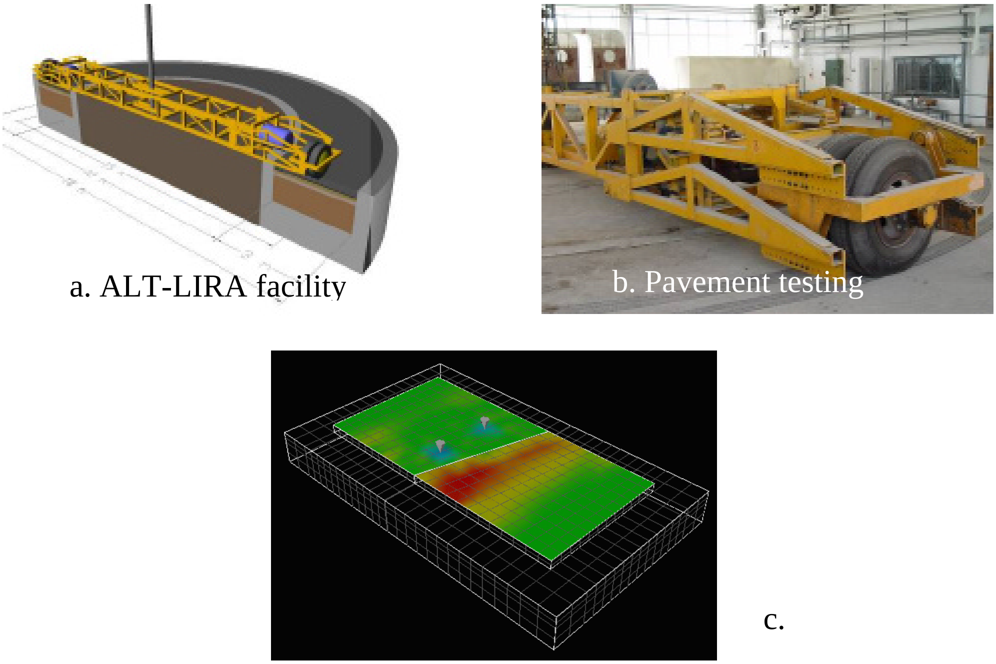


Figure 7 - Technical validation of concept of long lasting rigid pavements

3. STEEL FIBRE-REINFORCED ROLLER-COMPACTED CONCRETE

The main aim of EcoLanes work package 2 (WP2) is the development of SFR-RCC mixes, which have reduced energy requirements and use recycled materials, such as recycled aggregates and steel tyre-cord fibres, cement replacements (e.g. pulverised fuel ash), and low energy cements. To achieve this aim, WP-2 comprises the following five tasks.

- Reviewing use and design of steel fibres in concrete pavements to determine the ideal characteristics of steel fibres to be used in the SFR-RCC. For comparison purposes, industrially produced steel fibres are considered in addition to the steel tyre-cord fibres.
- Initial optimisation of the SFR-RCC mixes by considering various factors, such as cement type and content, aggregate type and content as well as fibre type and content. The main objective of this task is to determine the basic mechanical characteristics of each mix, such as compressive strength and elastic modulus.
- Flexural characterisation of the optimised SFR-RCC mixes by performing bending tests of prisms. The main objective of these tests is to asses whether the post-cracking resistance of SFR-RCC.
- Performing corrosion and chloride ingression tests as well as freeze-thaw tests with combined salt-stress. These tests started in early 2008 and their main aim is to assess the resistance of the developed SFR-RCC mixes to the main aggressive agents that surface transport infrastructure is normally subjected to.
- Development of models for the mechanical behaviour of the proposed SFR-RCC mixes by carrying out numerical analysis, including finite element and cross-sectional analyses. These models would then be used for the development of the LLRP concept and for the design of the four demonstration pavements.

3.1 Bending Behaviour of SFR-RCC

Bending tests were performed on rectangular prisms to evaluate the flexural strength characteristics (toughness) of the SFR-RCC mixes developed by WP2. Two types of recycled fibres, with different length distributions, were considered. Type A had a length range between 5-20 mm, while Type B ranged between 15-30 mm. Both types had an average diameter of 0.23 mm and a tensile strength of around 2000 MPa. In addition to the recycled steel fibres, two different types of industrially produced steel fibres were also considered: HE1/50 and BE1/50. The former is a loose cold drawn wire fibre with hooked ends, while the latter is a straight fibre with button ends (Figure 8). Both fibre types were 50 mm long, 1.0 mm in diameter and had a tensile strength of around 1000 MPa.

Figure 8 - Industrially produced fibres: (a) HE1/50 and (b) BE1/50

3.2 SFR-RCC Specimen Preparation

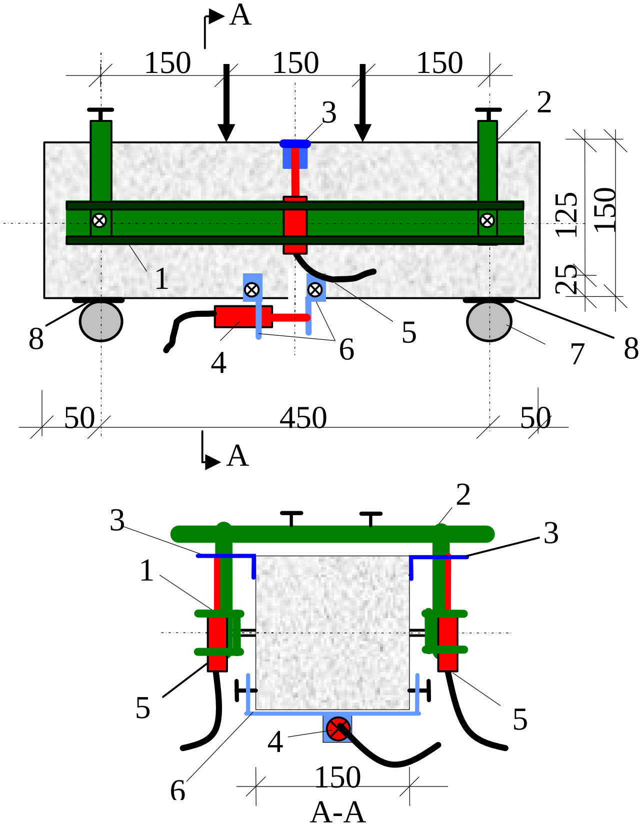
The SFR-RCC prisms were 150 mm deep, 150 mm wide and 550 mm long. Steel-plate moulds were used to eliminate the deformation of the moulds, caused by the severe external compaction. The specimens were cast in three layers and compacted by a suitable vibratory kango hammer. Following the recommendations of the RILEM bending test (RILEM, 2002), a day after casting, the specimens were demoulded and then placed in the mist room (+20°C and RH>95) until the day of testing. On the day of testing, a notch (25 mm high and 5 mm thick) was sawn at mid-span, into the tensile face of each rectangular prism (at a 90 degrees angle to the RCC layers), using rotating diamond blades. The purpose of the notch was to act as a crack inducer.

3.3 Testing Procedure

The testing of the notched prisms was carried out by following the recommendation of the RILEM bending test. It is noted that a four-point load arrangement was used instead of three-point load. The use of four-point load arrangement creates a region of constant moment and, hence, minimises the overestimation of bending resistance, caused at the point of load application by the load-spreading effect. The two supports and the device for imposing deformation consists of steel rollers with a diameter of 30 mm. Two rollers (one at the support and one at the device imposing the deformation) are capable of rotating freely around their axis and the longitudinal axis of the test specimen. All rollers are placed on steel plates (5 mm thick) to avoid local crushing of concrete and extraneous deformations.

Results from bending tests on concrete prisms are prone to significant experimental errors (due to spurious support displacements, machine stiffness and load rate) and, hence, extra care is required to obtain accurate deflection measurements (Copalaratnam and Gettu, 1995). To avoid these errors and the effect of torsion on the deflection measurements, a yoke was used as specified by the Japan Society of Civil Engineers (1994).

Average mid-span beam deflections were measured on both sides of the prisms using two transducers fixed to the yoke (LVDT5) and, hence, any torsional effects were cancelled out. One transducer (LVDT4) was mounted across the notch mouth to monitor the crack mouth opening displacement (CMOD), as illustrated in Figure 9. The SFR-RCC specimens were tested in a 100 kN servo-hydraulic machine under crack-mouth-opening-displacement control (CMOD). The machine was operated in such a manner that the CMOD was increased at a constant rate of 60 μm/min for CMOD ranging from 0-0.1 mm and 0.2 mm/min for CMOD from 0.1 mm until the end of the test.



1- Steel bar; 2- clamps with pins; 3- steel plate (glued to the prism); 4 - LVDT4; 5 – LVDT5; 6- clamps for LVDT; 7- supports, 8 - steel plates

Figure 9 - Set-up used for the bending test for RCC prisms

3.4 Bending Test Results

Figure 10 presents the bottom face of a cracked (in flexure) SFR-RCC specimen focusing on the fibre pull-out along the induced crack. The effect of fibre volume on the flexural behaviour of these specimens is shown in Figures 11 and 12. Figure

11 shows the flexural behaviour of specimens reinforced with industrial steel fibres; while, Figure 12 shows the flexural behaviour of specimens reinforced with steel tyre-cord fibres. It is apparent that an increase in the fibre volume, increases both the flexural strength and the residual strength following the initiation of cracking. The same trend was observed for all the different fibre types considered, with the industrial fibres being more effective.

Figure 10 - Fibre pull-out of a cracked SFR-RCC specimen

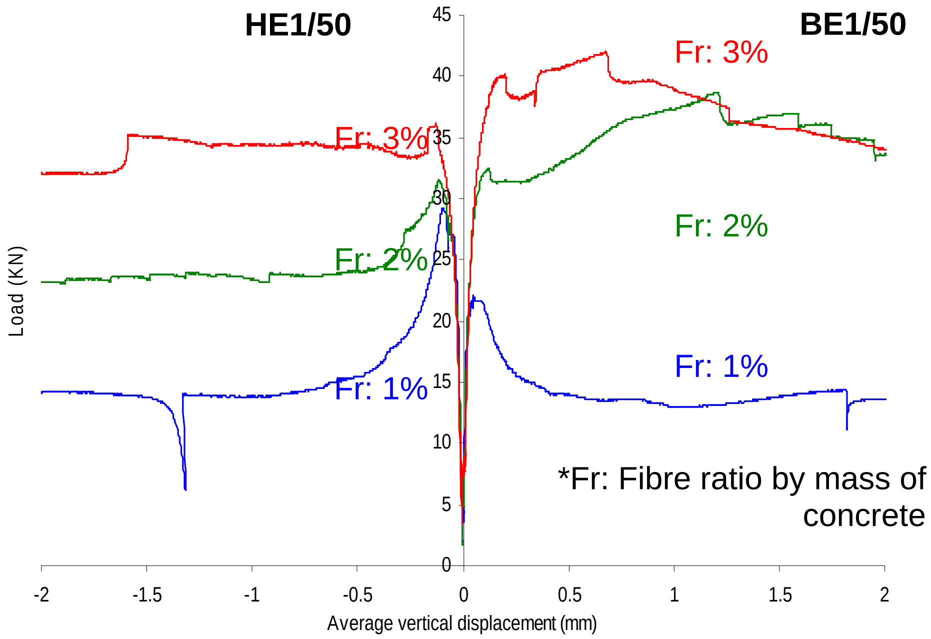


Figure 11 - Flexural behaviour of SFR-RCC utilising industrial fibres (3-day test)

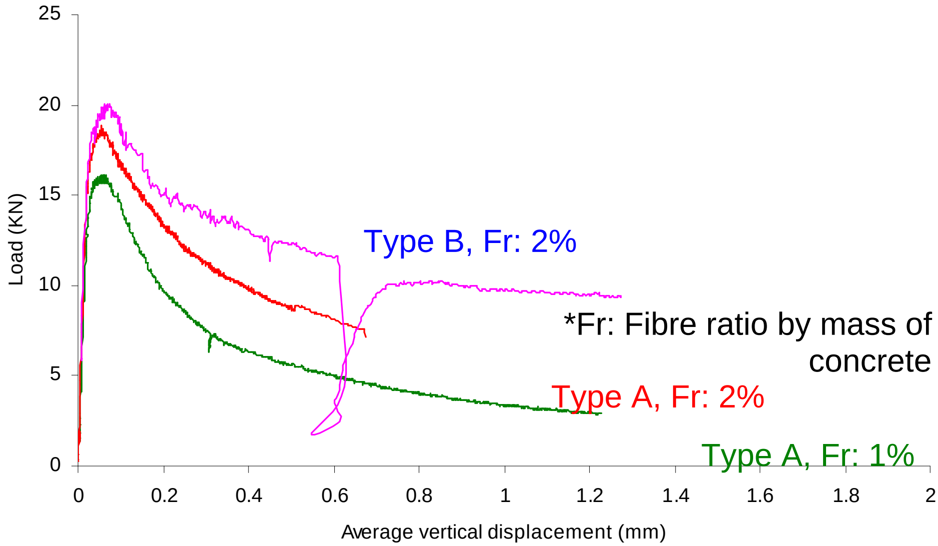


Figure 12 - Flexural behaviour of SFR-RCC utilising recycled fibres (3-day test)

The effect of fibre shape on the toughness can also be observed in Figures 11 and 12. Specimens reinforced with industrial steel fibres have higher modulus of rupture and present better post-cracking behaviour. This is largely attributed to the beneficial effect that the deformed ends have on mechanical bond, increasing SFR-RCC ductility. Flexural failure of the specimens occurred mainly due to fibre pullout. It is noted that, in specimens reinforced with the BE1/50 fibres, up to 50% of the fibres experienced fracture at their ends prior to pull out; increasing the energy absorption capacity of their specimens. BE1/50 fibre fracture is an indication of the high involvement of this type of fibres, demonstrated by the 'strain hardening behaviour', at fibre ratios higher than 2% by weight.

Figure 13 illustrates the positive effect of increased fibre length on the flexural residual load. Specimens with 50 mm long fibres exhibited an extended and more stable post-peak load- vertical displacement response, contrary to the limited vertical displacement obtained by the specimens with 5-20 mm and 15-30 mm long fibres (at low fibre ratios). A comparison of the two recycled fibre types (Fibre Type A and B) further reinforces this observation as the longer Type B fibres show a better post-cracking behaviour.

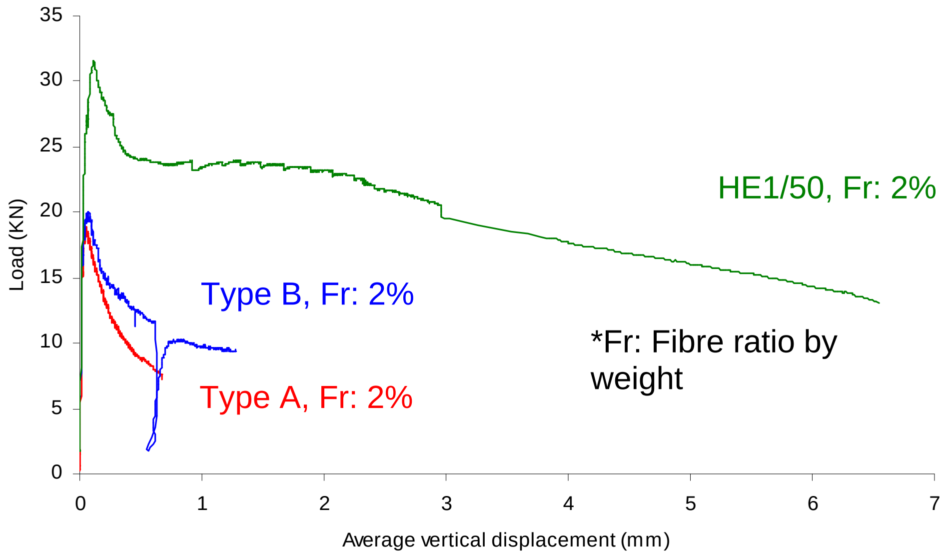


Figure 13 - Effect of fibre length on the flexural toughness (SFR-RCC) (3-day test)

4. CONCLUSIONS

Surface transport pavements are currently constructed using asphalt concrete, whilst Portland cement concrete is mainly used for heavy trafficked pavements. Often concrete pavements are reinforced with steel reinforcement to improve the concrete’s mechanical properties and reduce the pavement’s depth.

Despite the increasing prices of crude oil, the material cost of concrete pavements can still be higher than that of asphalt pavements and, hence, in order to provide a truly economical and sustainable solution for concrete pavements, it is necessary to utilise low-cost materials and to evaluate the energy cost as well as the cost of maintenance.

Research currently performed as part of the EU FP6 EcoLanes project aims to develop low-cost steel fibre reinforced concrete pavements by utilising roller-compaction techniques and recycled materials, such as steel tyre-cord fibres, cement replacements and recycled aggregates. It is expected that this construction concept will reduce the construction cost and time as well as the energy consumption in road construction.

Preliminary results obtained from bending tests of SFR-RCC prisms, carried out as part of the EcoLanes activities, have highlighted the importance of fibre geometry. The fibres examined from industrial sources resulted in better overall behaviour compared to the fibres recovered from post consumer tyres, at equivalent fibre ratios. However, if these fibres are mixed at higher fibre ratios they could be used as a viable alternative to the industrial fibres.

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The Influence of Changes in the Human Resource Management in a Knowledge-Based Economy

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Summary

The sustainable development is a modern concept of the knowledge-based economy. The need to work in an appropriate environment is a necessity, not a luxury in this century. All the physic and mental threats affect the efficiency of the human being. The globalization forces the countries to adapt to the new policies. The Human Resources Management is dealing with the organization's workforce. There are not good or bad people, only different ones. Along with the types, personalities and behaviours, the organization has to deal with multiple changes, as it functions in a dynamic environment. It is illustrated, briefly and with no intention of ending the subject, just some changes that have a great impact in the human resource structure, occupation, industry, demographic, pattern, social values changes. The economic environment is in a continuous change, the consumers' tastes, the employee's demands and the globalization bring new elements every day, so the organization has to be flexible and creative.

KEYWORDS: sustainable development, globalization, knowledge-based economy, changes

Motto:
"The XXI century success companies
will be those who will know how to hire, keep and develop the
knowledge of their personnel".
Lewis Platt,
CEO of Hewlett-Packard

1. INTRODUCTION

In a dynamic world, no organization, no matter how powerful it is, does have the reliability of its existence. The harder is the action of the environment on the small and medium enterprises. The managers are in the position to activate in an instable ground. The comfort of the `70s is now history. New politics and strategies have to be done in order to deal with this situation.

The organizations are open systems: they take their inputs from the environment, transform the inputs and obtain outputs (products and/or services). The main inputs are materials, human resources, financial, but the most important is the information (Figure 1). Nothing can be made without up-to-date information. This leads to knowledge, the only way one can hold the power of being informed.

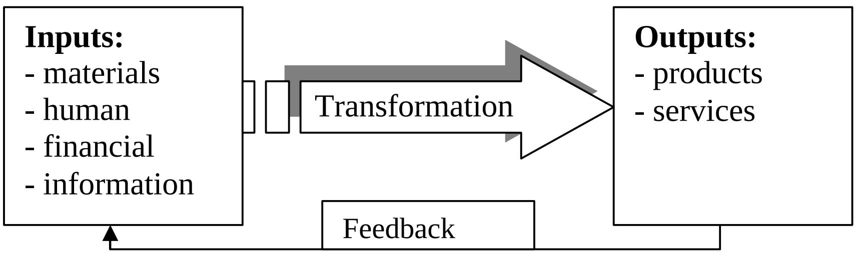

Figure 1. Organization as an open-system

The knowledge is not a modern concept. It comes from ancient times and it's connected to the desire of knowing and understanding the phenomenon. Aristotle said that "having knowledge about a matter means understanding its necessity". These are simple words but deep in their essence. And as long as the organizations are populated with people, they must be trained in order to meet the goals. In the long run, the specialists provide competitive advantage, according to their knowledge.

The organization, in a market economy, has to be adapted to the changing environment: legislation, technology, consumers` tastes, value systems. That's why the knowledge must be integrated in the organizational culture.

2. CONCEPTS, PRINCIPLES

Einstein said that curiosity is more important than knowledge. First step is to want to understand if someone is determined to do so, then comes the second step: learning in order to find out everything that can support the idea.

Learning and knowledge are key factors of success. In companies the battle field has moved from tangible resources (capital, lands, raw materials) to intangible resources (knowledge, abilities). Anyone can imitate the tangible ones (same suppliers), but no one can imitate the intangible ones. This is the way an organization can have the competitive advantage.

Knowledge is a strategic resource that generates sustainable development. A knowledge-based organization has some certain characteristics:

- decreasing physical activity in favor of increasing the knowledge base
- outsourcing the activities which are not essential for the organization in favor of the internalization of activities based on essential knowledge
- using the employees only for important jobs
- strategic development of the organization based on increasing the knowledge
- strong motivational system based on fair rewards

The knowledge-based management is crucial for a modern organization. It has some principles, almost general:

- deals with people and technology
- the implementation is made by knowledge-based managers
- is an endless process
- the intellectual propriety right is guaranteed
- needs training
- relies on technology
- it's a process, not a product

3. TYPES OF CHANGES IN A KNOLEDGE-BASED ECONOMY

Jack Welch, former CEO at General Electric, said: "You must be connected to change all the time. You cannot ignore reality, because anytime someone comes from another country with new products, or the consumers change their tastes, or technologies are changing. If you are not speedy and adaptable, you are vulnerable. These are real things for every segment in every business, in every country."

Today we are contemporary to the sustainable policies, adopted by all the countries of the United Europe. Romania has to change almost all of its concepts. In the communist regimes, people were used as any other inputs: no demands, no complains, no personal opinions, and especially no creativity.

The slogan “We work, we don’t think” was a fact. No one thought about the neighborhood, about the damages the factories cause to the environment. The same principles acted on the people: cold workplaces, unsafely conditions, long schedules.

The adaptation to a competitive market means the availability to change from the environment. And the changes heavily imply the human resources. Therefore, the human resources management deals with several changes:

a. Economic and workplace structure changes:

If until the '80s the main economic sectors were the industry and agriculture, now the statistics reveal that priority sectors are services and telecommunication.

But the biggest challenge is globalization. Opening the borders, not only for products/services, but for the workforce as well, brought a lot of companies in the situation to close their businesses because they could not face the competition any more.

In the new members of the United Europe, more than 30% of the companies disappeared after integration. It’s obvious that they were devastated by the ones which penetrated the market with their advantages, as the products’ quality, the brand, the price or the better paid workforce.

The United Europe tried to create a global market in order to reduce the imports from USA and Japan, even dictating restrictions. However, this didn’t happen, because the American and Japanese companies opened subsidiaries in Europe and began production there, just to annihilate the restrictions.

Therewith, globalizations imposed changes in the workforce structure: the multinational companies come with their own structures, they train the managers in the home countries, and so the national specific is diluted.

Concerning the International Human Resources Management, the multinational companies have to deal with the big geographic distances between the home country and the country they have business (Table 1).

That’s the reason why new kinds of personnel were born:

- expatriates, or the multinational company’s employees who activate in their born country

- employees from the company’s home country

- employees from a third country, different from the born or home country.

Companies as Bechtel, Coca-Cola, Eastman Kodak, use this modern system to distribute their personnel, both the operative and the leading one.

Table1. Advantages for hiring personnel from other country than the multinational company's home country

Born country's employees	Expects	Third country`s employees
-small costs -are familiar with the culture, the environment -speak the language -lack of legislation barriers	-experience in the born country -mobility -accessible control -promotion from inside	-international experience -facile adaptation in an international environment -speaking more languages

b. Changes in the occupations and industry structures:

As mentioned, today's trend in the companies' activity is for more services and less production. The workforce has to find new specializations, to give up the occupations they were prepared for, because today the unemployment is a mass phenomenon. A real problem comes from the fact that the former communist countries that faced the politic changes don't have funds for a serious social protection. The only solution will be the prequalification.

But there is an almost scarified generation: the people over 45-50, and for those it is very hard to learn a new occupation, and even so, it is difficult to find a new job, as long as there is an age limit on hiring.

New technologies substitute the risky or routine work: for example, the workforce is no longer exposed to toxically environment; the robots do that. So is for the repetitive tasks. There is an interest on creativity and resourcefulness, and these will never become boring, but will bring satisfaction and pride for those who are active involved.

There is a raise on the importance of modern industries: telecommunications, computers, optic fibre, and laser. They will need super-qualified personnel.

This kind of change is perceived in Romania as well. New occupations are rising, and the best example is that the CAENE code is filled in with 97 new occupations. The working population has to line up with these modern demands of high qualification, but they need information, then education and specialization to fulfil.

c. Demographic and workforce structure changes:

There are modifications in the demographic structure of the world population. The number of hired women grew up. The traditional pattern has also changed. Women, who are employees, wife and mothers, do not have the physical men's force and they cannot be hired according to the old practices. This fact cannot be ignored, especially on compensations. The surveys show that the women's wages are 20% smaller than the men's.

So, the employers will be forced to create conditions in which women will perform: child care systems, flexible schedules.

Another important fact is the willing of the younger to be financially independent. They seek for jobs in services, but in the army as well. They are a social category that must be protected.

Romania understands these modern trends: the workforce exchange for students is full with applications. And if, till now, America was the “land where dreams come true” for every student who wants to work in any job, even unqualified, now they prefer to work in any place in the world, as long as they are well paid.

A good example is the Workforce Student Exchange in Cluj-Napoca in March 2006, concerning 600 jobs in Alaska: catching, evisceration, honking and loading it in boats. The schedule was 16-18 hours/day, but wages were great. It is not the only way to use the willing to work of the younger: in USA there is a group of 600 companies, naming only Chevron, Monsanto, AT&T, which recruit students with potential and they hire them during holidays, and then attract them to the company.

Not only the younger take benefit from the employers’ attention. For example, Day Inn Hotels chain recruit retirements, because they noticed that these have a little rate of absenteeism and work accidents, and have more satisfactions in work.

<u>d. Changes in patterns of work</u>

Along with the structural and demographical changes, there came obvious changes in the patterns and the life style of the population.

“Time is money” says the American pragma. There is no minute to lose. Money offers power and comfort. So, many people prefer workplaces in order to fill the financial blanks of them and their families. Temporary employees are in trend now. This is a win-win case. It is good both for the employee, and for the company which doesn’t need to hire numerous personnel to deal with peak times, and then to pay them in relaxed times. This aspect comes from the fact that economic sectors have changed. There are crowded periods in services and in agriculture as well.

Another crushed “myth” is the immobile schedule, from 9 to 17, from Monday untill Friday. The companies understood that the employees need time for personal problems, so they created a flexible work agenda with a fix schedule work time of day and a flexible one, of course respecting the 8 hours program. The effect is lower absenteeism and tiredness and grater work satisfaction.

<u>e. Changes in social values</u>

At last, but not at least, the Human Resources Management has to deal with the social values change. The sexual minority discrimination is not permitted in a modern business environment. The famous picture “Philadelphia” shows the fact that the companies are not allowed anymore to fire their employees for their sexual preferences, as long as their behaviour does not affect their competences and they don’t aggress the rest of the workers.

Sexual harassment at the workplace is something that deals with the equal treatment of the employee and it is against the law. In 1995, there were 15.549 cases in the world, and 10% were complained by men.

Another modern method is the anti-drug test before hiring, a situation no one could imagine in the 80’s, but now drug using is already a worldwide social problem.

4. CONCLUSIONS

Peter Drucker gave a very concentrate definition of the knowledge-based organization: “this is the XXI century organization and therefore it has the following characteristics: specialists, flat hierarchical pyramid, co-ordination by non-authoritarian methods”. The Human Resources Management is an important pillar to the modern concept of sustainable development. The human being is present in all the companies` departments: management, supply, maintenance, accounting, executive. The way they understand, appreciate, love and do their jobs influence the company, the field and in the end the whole national economy. Their devotion and trust in this policy can raise or can bury the theory. As Abraham Maslow said years ago, the safety needs are essential. An individual can’t climb the pyramid of needs unless the primary ones are not fulfilled. The knowledge-based organization has to deal with questions like: “how?” or “why?” In order to answer these simple but profound questions, the modern companies must be innovative (to create new knowledge), learning (to assimilate new knowledge) and interactive (to cooperate with the economic environment).

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Steel-Fibre-Reinforcement and Increasing the Load-Bearing Capacity of Concrete Pavements

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Abstract

Elastic theories form the basic concept used in most of the existing design codes for industrial or transportation plain and conventionally reinforced concrete ground slabs. The post-cracking load bearing capacity of slabs-on-ground is not taken into account in most of these codes. Therefore, these codes (e.g. PCA, ACI) cannot be used directly for steel fibre reinforced concrete (SFRC). Guidelines for SFRC (e.g. Concrete Society) use the ultimate limit state concept for fibre reinforced ground floors, but only partially, since cracking is only allowed to occur on the bottom surface of the slab. The highly repetitive nature of the loads which may cause considerable degradation in the mechanical properties of the pavement and foundation also is not considered in this method. The aim of this paper is to evaluate the load-bearing capacity of SFRC pavements through numerical simulations, and to assess the accuracy of the analytical methods used in different design codes,

Keywords: SFRC, numerical analysis, load-bearing, concrete pavements.

1. INTRODUCTION

Due to considerable differences between the strength of concrete in tension and compression, the tensile strength of rigid pavements usually dominates the design. Therefore, in plain concrete pavements, the compressive capacity of the slab remains largely unused. The material in general remains in the elastic domain until cracking takes place, hence, its behaviour could be predicted by elastic analysis.

For reinforced concrete (RC) pavements a significant part of load bearing capacity is developed after cracking and mobilization of the force in the reinforcement. Therefore, the slab enters the non-linear domain of structural behaviour and using elastic assumptions leads to a significant underestimation of the slab capacity. SFRC pavements behave similar to reinforced concrete, though the effective amount of reinforcement is much less, and hence also require non-linear analysis for predicting their behaviour.

Existing design codes, for industrial or transportation slabs, rely on elastic or basic elasto-plastic theories and methods of analysis. This paper begins by a review of the various theories used. However, these methods are not perfect for general analysis and numerical methods are more flexible in the range of problems they can solve.

Load bearing capacity of concrete pavements may be reduced due to considerable degradation in the strength properties of the material caused by fatigue effects. Steel fibres increase the fatigue resistance of concrete significantly (ACI 544.4R 1999). However, the effect of fatigue on SFRC pavements has not been sufficiently studied in the published literature and requires comprehensive research.

The elastic and inelastic behaviour of SFRC pavements are studied in this paper using FE models. The load-bearing capacity of SFRC pavements is evaluated for a typical road slab, considering the issue of fatigue. The results are compared with analytical methods. In the absence of test results for investigating the effect of fatigue on SFRC pavements, at this stage, the fatigue effect is simulated by a reduction factor in the strength of concrete.

2. EXISTING THEORIES FOR ANALYSING RIGID PAVEMENT

The first complete design method for rigid pavements was developed around 1920, based on Westergaard's theory (ACI 360R, 1997). Westergaard assumed that the slab is a homogeneous, isotropic and elastic material resting on a perfect subgrade. Westergaard's equations are still widely used for computing stresses in pavements and validating models developed using different techniques, In spite of overestimating the required slab thickness. In 1943, Burmister (1943) proposed the theory of stresses and displacements in layered systems. This theory was never developed enough for engineering design practices, because it is not applicable for limited-length slabs under edge and corner loads. Later, Losberg (1961) and Meyerhof (1962) developed similar strength theories based on the yield line concept. However, these theories are not able to predict the deformational behaviour of the slab-foundation system.

The classical differential equations are often used to predict the structural behaviour of rigid pavements. Solving these equations by conventional methods is feasible only for simplified models with homogeneous materials and continuous geometry for slab and subgrade. Therefore, the use of this approach for a real rigid pavement, which may contain discontinuities and be supported by a non-uniform subgrade, is quite limited. However, it is possible to use them as a bench-mark for validating other numerical models, by comparing the results under similar assumptions. Considering an infinitely extended plate carrying a load "P" which is distributed over a small area, the maximum tensile stress can be computed by closed-form equations. In this case, Timoshenko (1952) developed equations, using the same assumptions as in the Westergaard's theory. In the case of a highly concentrated load, Timoshenko's equations corrected by means of the thick-plate theory using Westergaard's equations.

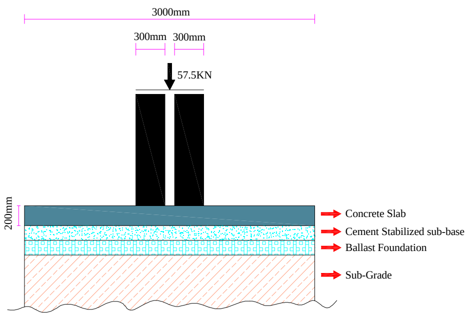
In the existing theories two models have been used for the subgrade: Winkler (or dense liquid) subgrade and elastic-isotropic solid subgrade (ACI 360R, 1997). Winkler foundation is assumed to deflect under an applied vertical force in direct proportion to the force, without shear transmission to adjacent areas of the foundation. In the elastic-isotropic solid model the applied load to the surface of the foundation is assumed to produce a continuous basin. The elastic response of real soils is located somewhere between these two extremes.

3. NUMERICAL METHOD

A good numerical tool for analysing the behaviour of rigid pavements is the "Finite Element" (FE) method. The FE method offers good potential to solve complex problems and can be used to analyse rigid pavements having many different boundary conditions. Two different approaches are examined for analysing a concrete pavement carrying a single concentrated load. In the first approach, materials are assumed to behave elastically. In the second, the non-linear behaviour of concrete is considered and the smeared crack model is used for simulating the post-cracking behaviour of the SFRC pavement. Winkler model is used for simulating the foundation. The ABAQUS finite element package (Hibbitt, Karlsson and Sorensen 2004) is used to perform the analysis.

The chosen slab to be analysed in one that will be tested as part of a large European funded project called "Ecolanes". The slab will be subjected to accelerated load testing at the TUI, Romania. A general description of this test is summarised in Figure 1 as well as in Table 1.

(a)



(b)

Figure 1 (a) Circular accelerated loading test, TUI Romania (b) Layers and the loading

Table 1 Slab details

Slab:	Track width	: 3.00 m
	Slab thickness	: 200 mm
	Elastic modulus	: 32GPa
Load:	Double wheel load	: 57.5 kN
	Position	: Moving along the centre line
	Number of load cycles	: 1.5 million
Supporting layers:	Equivalent modulus of reaction	: 0.4 N/mm^3

Two models were analysed: (1) Infinite slab, to compare the results with existing closed form equations (A slab size of 6×6m was found to sufficiently simulate a large slab). (2) Finite width slab of 3m (A 3×6m slab was found to be sufficient in this case).

Shell elements were used for modelling the slab, in which the thickness is significantly smaller than the other dimensions. Shell elements have displacement and rotational degrees of freedom at each node.

3.1 Reliability Assessment and Mesh Sensitivity Analysis of the Elastic FE Model (Infinite Slab)

4 element dimensions were used for mesh sensitivity analysis, to compare the results and to evaluate the sensitivity of the model to mesh refinement. The results of FE analysis for an infinitely extended slab are compared with two closed form equations (Table 2).

Figure 2 shows the percentage difference between the maximum tensile stresses calculated by FE analysis and closed form equations, for different element sizes. This figure clearly shows the sensitivity of the model to the element size. Using finer meshes, the numerical solution tends to converge and get closer to the results of the closed-form solutions.

Table 2 Element dimensions and results of elastic analysis for an infinite slab

Method	Mesh No.	Element Size (mm)	Maximum tensile stress at the bottom face (MPa)
FE Analysis	1	300	0.711
	2	150	1.159
	3	50	1.339
	4	25	1.363

Closed-Form Solution	Timoshenko’s equation	1.36
	Westergaard’s equation	1.423

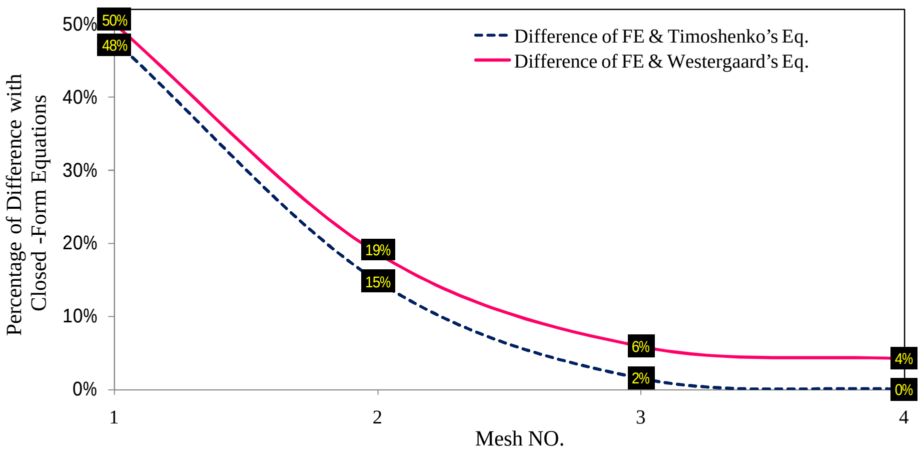


Figure 2 FE elastic analysis, compared with closed-form solutions

The results indicate that the developed finite element model used is conceptually reliable and can be used for analysing other slab geometries and loading configurations. Due to acceptable accuracy, 50 mm element size is used for future analysis.

3.2 Non-Linear FE Analysis of the Finite Width Slab

FE analysis of the slab was performed under the service load, (double wheel load with two contact areas, each 110×300mm) which was applied centrally on the slab. Under this load the stresses created in the slab were much less than the cracking stress. Therefore, the load was increased gradually, to monitor the post-cracking behaviour of the slab until complete collapse. The post-cracking tensile strength of SFRC increases the load bearing capacity of the structural member beyond the plain concrete capacity. The tension softening diagram of concrete, which represents the relationship between tensile stress and tensile strain in the fracture zone, describes the post-cracking behaviour of concrete. Figure 3 shows the concrete material properties used in the analysis. The ratio of ultimate biaxial compressive stress to ultimate uniaxial compressive stress (Kupfer et al. 1973) was assumed to be 1.15. The approach used for simulating the non-linear behaviour of the concrete is the smeared crack approach.

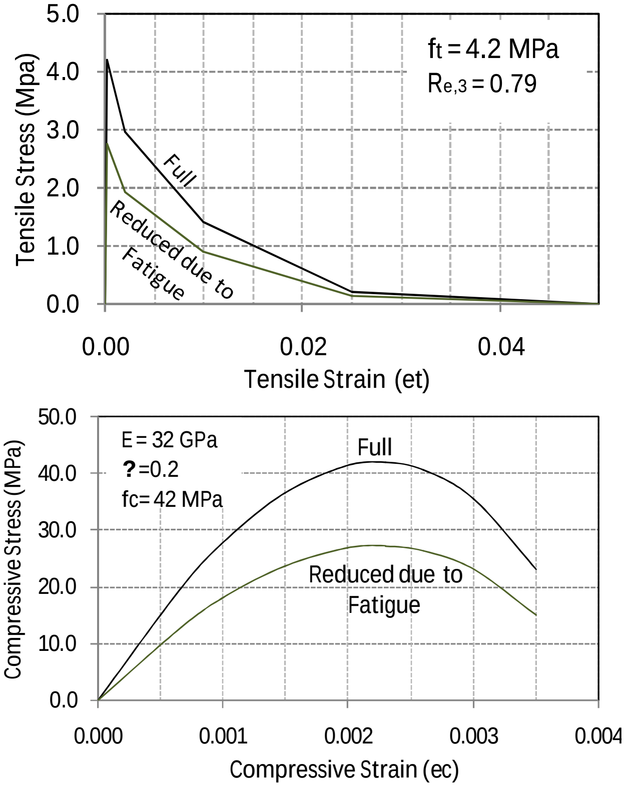


Figure 3 Concrete material properties

As the load increased, the flexural stresses at the bottom of the slab reached the flexural strength of the concrete, leading to a transversal tension crack caused by positive moments. Further increases in the load led to a longitudinal tension crack in the bottom of the slab. After that further increase in the load did not cause further increase in positive moments, but the moments were redistributed. This resulted in a substantial increase in circumferential negative moments some distance away from the loaded area, resulting in circumferential tensile cracks at the top of the slab. The ultimate load occurred when the transversal crack reached the edges of the slab which was split into two. The load versus central displacement curve of the slab is presented in the next section, Figure 4.

3.3 The Fatigue Effect

Due to traffic and cyclic environmental conditions concrete pavements are subjected to repetitive loading. Generally, fatigue strengths for SFRC are 65 to 95 percent at one to two million cycles of non-reversed load, as compared to typical values of 50 to 55 percent for slabs without fibres. For properly proportioned high-quality SFRC, a fatigue value of 85 percent is often used in pavement design (ACI 544.4R 1999).

To account for the fatigue effects in numerical analysis in this paper, the material capacity was reduced to 65% of the static strength, and the structure was analysed as for static loading. Figure 4 shows the load-displacement curve at the centre of the slab with and without consideration of the fatigue effect. The loads corresponding to initiation of transversal, longitudinal and circumferential cracks are illustrated on the curves.

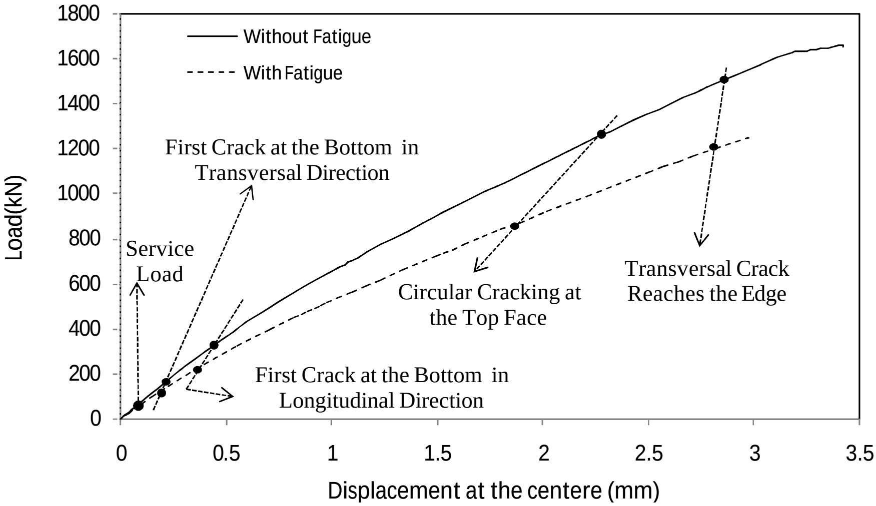


Figure 4 Load-displacement curve at the centre of the slab.

Results of the numerical non-linear analysis are compared with the non-linear method of the Concrete Society (2003).

4. CONCRETE SOCIETY METHOD FOR SFRC SLABS

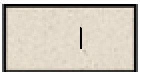
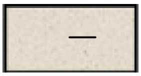
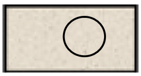
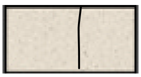
The method presented in TR34 of the Concrete Society (2003) for fibre reinforced ground floors is based on the elasto-plastic theory developed by Meyerhof (1962) using the ultimate limit state concept, but only partially, since cracking is only allowed to occur on the bottom surface of the slab.

Using the Concrete Society method for dual point loads acting centrally, the total collapse load can be approximated. According to the Concrete Society definition, collapse load is reached immediately before the development of visible circumferential cracks on the top of the slab. In this method, the contribution of the subgrade reaction is ignored. Using the above calculations and ignoring the partial safety factor, the ultimate load of the slab with the characteristics presented in section 3, is estimated at 790kN. This is the maximum load carried by the slab immediately before the cracks form at the top face.

5. DISCUSSION

A comparison of results for each crack mode and corresponding loads is shown in Table 3. For fatigue loads, this table shows that the load corresponding to the first crack reduces proportionally to the reduction in material property. Deterioration has less impact on the final capacity.

Table 3 Loading capacity, with and without considering fatigue effect

Cracking stage		Numerical Model			Concrete Society
		Central load (kN)		Load bearing ratio (Fatigue/No fatigue)	Central load (kN)
		No fatigue	With fatigue		No fatigue
First transversal crack at the bottom		175	115	66 %	-
First longitudinal crack at the bottom		320	215	67 %	-
Circumferential crack at the top face		1250	850	68 %	790
Cracking all over the transversal direction		1500	1200	80 %	-

6. CONCLUSION

Analysis done using FE shows that the slab capacity exceeds the elastic capacity and service load by many times. Therefore, the approaches used in most of the codes for the design of transportation slabs, which are based on classical elastic theories, are not suitable for SFRC pavements.

Numerical analysis showed that the first crack is expected in the transversal direction followed by cracks in the longitudinal direction. Top cracks around the loading surface occur at much higher loads and the slab eventually fails by splitting into two.

The effect of fatigue could be taken into account by reducing the material properties and this has a greater impact on the first crack load than the ultimate capacity.

A comparison with the Concrete Society method for ground slabs shows that more work needs to be done to bring the two approaches together.

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Correlation Of The Accelerated Traffic With That Encountered On Real Current Roadway

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Summary

The pavement experiments with Accelerated Load Testing (ALT) facilities, considered as an intermediate stage between the laboratory and the in-situ studies , are justified only when a sufficient confidence level for the equivalence value of the accelerated traffic with the real roadway one is assured , besides the specific elements necessary for the pavement modeling .

The ALT experiments , conducted during significant stages of behavior of pavement structures, characterized by specific technical condition indexes, could provide an objective character for the programme prioritization of road maintenance /rehabilitation works.

In this context, the paper presents a proposed ALT-RLT correlation valuable for the circular track facilities, customized to the - Accelerated Testing Facility ALT-LIRA existing at Technical University " Gh. Asachi" Iasi, in Romania. A comparative study for the equivalence of physical vehicles in standard ones (ESAL of 115kN) using average equivalence coefficients, based on traffic agressivity criterion, specified by norms [1] and the proposed calculation based on design criteria established for road pavements in accordance with actual the analytical design method is conducted in the frame of a case study, involving three significant types of road , namely one European and two Nationals : a principal and a secondary one.

Keywords: Accelerated Load Testing-ALT, Real Load Testing -RLT, Standard Axle Load- SAL, equivalence coefficients, circular track, pavement design

1. INTRODUCTION.

The undertaking of experiments with Accelerated Load Testing (ALT) facilities, as an intermediate stage between the laboratory and the insitu studies, is justified only if the following conditions are assured:

- The traffic is simulated with representative parameters for the standard axle load, the form/dimension of the tire print, unit pressure at the contact surface, the speed of application of load;
- The establishment, through the equivalent traffic, of the correlation between the accelerated traffic and the real one in the current way, in the conditions of eliminating empirical equivalence coefficients, in order to ensure an appropriate confidence level;
- The experiment is conducted on significant stages of behavior of road pavements, evaluated through specific technical condition indexes. Thus, in the conditions of the ALT-RLT correlation, an objective character of the intervention steps is assured during the use of the road.

2. THE EQUIVALENCE OF THE SIMULATED TRAFFIC WITH THE REAL ROAD WAY TRAFFIC

In the conditions mentioned above, the following relation for calculation of the equivalence of the real roadway traffic and the simulated traffic on ALT circular facilities could be derived:

$$n.N_s.j.\frac{n_1}{n_2}=\sum_{i=1}^{n}\frac{N_{io}(1+b_{ti})}{2h_i}a_i.\mathrm{E}_i.365.t \tag{1}$$

Where:

? – number of arms of the traffic simulation facility; ? =1…4

N_s –number of rotations of the arm

F - ratio between the applied load through simple / twin wheels and the load of standard vehicle (a safety value f =1,0 is recommended, in order to limit the process only for current roadway traffic)

$?_1$/ $?_2$ - ratio considered in case of facility’s translation during traffic simulation, where:

$?_1$ = number of passes of simple/ twin wheels on half on the most trafficated strip on circular facility;

$?_2$ = number of passes of simple/ twin wheels, through a cross profile of the most trafficated strip on the circular facility;

The traffic on the same path presents as advantages:

- Reduction of testing duration;
- Modeling the formation of longitudinal rutting in pavements, any time, function of the pavement type and temperature

N_{io} - number of physical vehicles in the “i” group, in current way, during the “ year zero” (the opening of road pavement to traffic), obtained from the partial/general traffic census $ß_{ti}$ - evolution coefficient for physical vehicles in the “i” group, at the pavement service life: “t”;

a_i - the proportion of Ni, for the most traffickated direction;

E_i - equivalence coefficient, expressed in number of standard vehicle (ESAL) for physical vehicles of the group “i”;

?$_i$ - number of passes of physical vehicle from the group “i” for a superposition of the tire print. In the absence of these and for preliminary studies the values from norms [1] may be used ; approximate values, (+/_5%) are considered in the case study calculations (pt.5) . This coefficient is established , based on appropriate insitu studies.

t - the service life (expressed in years) ; 365 – the number of days in the year;

3. THE ACCELERATED TESTING FACILITY ALT-LIRA FROM TECHNICAL UNIVERSITY “ GH. ASACHI” IASI .

As the case study (pt.5) refers to the Accelerated Testing Facility ALT-LIRA from Technical University “ Gh. Asachi” Iasi , the main characteristics of this facility are briefly presented [2]:

- The circular vat , made of reinforced concrete has the following dimensions: 15m diameter , 3 m width and 2 m depth. The pavement structures are built up inside the vat , on an earth layer of over 1,2 m depth. A draining layer of 0,2 m depth, at the vat bottom, contains the pipes for water feeding, in order to simulate the modification of the water table level.
- The accelerated pavement testing facility consists of a steel arm , 15 m long (? =2). Each end of the arm is elastically supported by multi-leaf springs on a system of dual wheels, equipped with standard truck tires. Even the wheel speed can reach up to 40 km/h, during the ALT testing it is limited to 20 Km/h due to safety reasons. The equipped arm weights 115kN (each dual wheel transmits a concentrated moving load of 57,5kN). This is representative the standard axle load according the Romanian standards,

The whole facility is located inside of an appropriate building, so that the temperature conditions could be controlled, ranging from +16^0 C during the winter to + 35^0 C during the summer. In order to simulate negative temperatures similar to those natural ones, an air conditioning device, covering one quarter of the total length of the circular track , for cooling the pavement surface to minus 20^0 C is also available.

4.THE EQUIVALENCE COEFFICIENTS OF PHYSICAL VEHICLES IN STANDARD VEHICLES

4.1. In Romania the structural design of flexible/ semi-rigid pavements is done through an analytical method [3], characterized by the following considerations:

- the calculation scheme is represented by a Burmister multi-layers type ;
- total adherence is considered at the layer interfaces;
- the standard vehicle is represented by the standard axle load (ESAL – 115 kN);

The equivalence coefficients of physical vehicles in ESAL of 115 kN are calculated based on the traffic agressivity criterion[4],[5], taking into consideration average values established for the national road network. (Table 1)

Table 1. The equivalence coefficients of physical vehicles in standard vehicles (ESAL =115kN)

The group of vehicles	Trucks and similar with		Articulated vehicles with semi-attachments	Buses	Trucks with attachments (road train)	Comments
	2 axles	3, 4 axles				
Equivalence coefficients	0,4	0,6	0,8	0,6	0,8	Average values (pt.4.1)
	0,87	0,93	0,86	0,98	0,93	According pt 4..2
?$_i$	2,64	1,93	1,93	2,37	1,93	
	2,5	2,0	2,0	2,50	2,0	According pt4..2

4.2. The use of the analytical method for structural design criteria [3] for the equivalence of traffic, permits to derive the equivalence coefficients from the following ratios :

$$e_{r-f} = e_{r-} \text{ ESAL} \quad (2)$$

$$s_{r-f} = s_{r-} \text{ ESAL} \quad (3)$$

$$e_{z-f} = e_{z-} \text{ ESAL} \quad (4)$$

where :

e_r - the specific tensile horizontal deformation at the bottom of the bituminous layers;

s_r - the tensile horizontal stress at the bottom of granular layer / stabilized aggregate with hydraulic / puzzolanic binders layers;

e_z - the specific compressive vertical deformation at the level of subgrade layer ;

e_{r-f}, s_{r-f}, e_{z-f} - are corresponding to the physical vehicle;

$e_{r\_}$ ESAL, $s_{r\_}$ ESAL, $e_{z\_}$ ESAL – are corresponding to the standard axle

4.3. The coefficients : e_r s_r e_z are calculated for the semi-axles of the physical vehicles, considered for the various groups of vehicles (Table 2)

Table 2. Loads on the vehicle semi-axles

The group of vehicles		Loads on semi-axles (kN)			Distance between axles
		Simple	Twin	Tridem	
Trucks and similar with :	2 axles	40	-		-
	3, 4 axles	-	2x 45		1,30
Articulated vehicles		-	-	3x40	1,40
Buses		50	-		-
Trucks with attachments (road train)		-	2x45		1,30

5. THE CASE STUDY . ESTIMATION OF THE DURATION OF THE ALT EXPERIMENTATION OF A SEMI-RIGID PAVEMENT STRUCTURE, FOR EUROPEAN (E), PRINCIPLE NATIONAL , AND SECONDARY NATIONAL ROADS .

The semi-rigid pavement structure selected for this case study is described in Table 3

Table 3. The description of the semi-rigid pavement structure

The type of road layer	Material used for construction	Layer thickness (cm)	E- Resilient Modulus (MPa)	μ- Poisson ratio
Wearing course	Asphalt concrete	4	3600	0,35
Binder course	Asphalt mix (HMA)	5	3000	0,35
Base course	Asphalt mix (HMA)	8	5000	0,35
Foundation up-layer	Cement -bound graded aggregate	23	1200	0,25
Foundation bottom-layer	Ballast	25	750	0,27
Capping layer	Ballast	15	135	0,27
Subgrade	Silty soil	-	70	0,35

5.2.The Average Daily Traffic (ADT) values for the years 2005 and 2020, for those three categories of roads considered in the study are presented in Table 4.

Table 4. The Average Daily Traffic (ADT)

Road category	The group of physical vehicles														
	(1)			(2)			(3)			(4)			(5)		
	ADT 2005	ß$_{15}$	ADT 2020	ADT 2005	ß$_{15}$	ADT 2020	ADT 2005	ß$_{15}$	ADT 2020	ADT 2005	ß$_{15}$	ADT 2020	ADT 2005	ß$_{15}$	ADT 2020
E	469	1,99	933	253	1,55	392	557	1,47	819	465	1,96	911	93	1,35	126
NR-principle	365	1,65	602	264	1,63	430	283	1,50	425	282	1,40	395	155	1,50	233
NR Secondary	125	1,58	198	78	1,31	102	149	1,36	203	10	1,22	12	15	1,77	27

5.3. The ALT loading duration on the circular track

In Table 5 , the following parameters are presented:

- the total traffic, expressed in ESAL of 115 kN, for the period 2005 – 2020;
- the values of the a_i coefficients , according the Romanian technical norms , function of number of traffic lanes;

- number of necessary rotations of the traffic simulation facility;
- number of effective days for a work program of 5 days/ week, 12 hours/ day; traffic on the same path , speed V= 20 km/h

For the various stages of measurement of the surface/ structure characteristics, a supplement of 2..5% from the total number of days specified in the Table 5, is recommended to be taken into consideration.

Table 5. The ALT loading duration on the circular track

Road category	The total traffic (ESAL= 115kN)		ai	Ns rotations		No. of effective days (Total number of days)	
	Ei (averages)	Ei (pt. 4,2)					
0	1	2	3	4	5	6	7
E	1.713.675	2.491.125	0,45	856.838	1.245.563	168 (235)	244(342)
NR-principal	1.324.950	1.927.200	0,50	662.475	963.600	130(182)	189(265)
NR Secondary	399.675	514.650		199.838	257.325	39(55)	51 (71)

6. EVALUATION OF TECHNICAL CONDITION OF FLEXIBLE/ SEMI-RIGID PAVEMENT STRUCTURES

6.1. According the Romanian norms [6],[7] the global distress index IG, calculated with the relation 5 , is considered:

$$IG = \sqrt{IEST - IESU} \tag{5}$$

where :

I.E.ST – the structural (evaluation) index

I.E.SU – the surface condition (evaluation)index

The evaluation of the distress condition of pavements is given in Table 6

Table 6. The evaluation of the distress condition of pavements

The global distress index	Very Good	Good	Mediocre	Poor
IG	>95	90...95	77...90	<77

6.2. The real traffic, in the current way, which corresponds to the **Ncr** - values , for which , on the circular track, critical values for the technical condition indexes are recorded, is obtained with the relation 6 (equation of second order) where the only unknown parameter is “x” the duration of the real traffic.

$$n.N_{cr}.j.\frac{n_1}{n_2} = \sum_{i=1}^{n} \frac{N_{io}(1+b'_{ti})}{2h_i} a_i.\mathrm{E}_i.365.x \qquad (6)$$

where :

$$b'_{\mathbf{ti}} = \left(b_{\mathbf{ti}} - 1,0\right) / \ \mathrm{t} \qquad (7)$$

?. . F . N$_{i0}$, ?$_1$/ ?$_2$, E$_i$, ?$_i$, ß$_{ti}$, t , have the significance given for relation (1)

6.3. The '”**x**” values (years) are permitting the programming of the various stages of insitu interventions.

7. CONCLUSIONS

7.1. The relation (1) used for the calculation of the number of rotations of a circular traffic simulation facility on a circular track , equivalent to the real current way traffic, takes into consideration the parameters involved in those two categories of traffic.

7.2. The assurance of a standard axle in the frame of ALT investigations improves the confidence level of the study, the equivalence coefficients being used only for physical vehicles.

7.3. The homogeneity of the criteria for structural design of pavement structure and for the calculation of the equivalence coefficients of physical vehicles in standard/ etalon vehicles was found to be opportune.

7.4. The relation(6) assures an objective character for the programming of various stages of intervention , during the design life of the pavement structures.

7.5. Specific studies concerning the distribution of the real traffic in the road cross profile , as function of the various groups of physical vehicles , are justified for the definition of the ? parameter for the superposing of the vehicle tire prints [8],[9].

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Specific features of structural design of Long Lasting Rigid Pavements – LLRP for demonstration projects located in different climatic regions in the frame of the European EcoLanes research project

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Summary

This paper presents the specific features of structural design of Long Lasting Rigid Pavements – LLRP for some of the demonstration projects located in different climatic regions in the frame of the European EcoLanes research project.

Keywords: concrete pavement, rigid-composite pavement, pavement design, accelerate load test - ALT, real load test – RLT

1. INTRODUCTION

This work is part of the EU collaborative project EcoLanes [1] funded under the priority thematic area of Sustainable Surface Transport in the 6-th Framework Programme of the European Community, which aims to develop Long Lasting Rigid Pavements- LLRP. This project, coordinated by University of Sheffield, draws expertise from six European countries and its Consortium comprises three universities, two industrial partners, the European Tyre Recycling Association-ETRA and three end-users. The main objective of this project is the development of pavement infrastructure for surface transport using classic/ conventional and the roller compaction technology in combination with concrete mixes reinforced with steel tire-cord fibres, recovered from post-consumed tires, seeking significant benefits expressed in terms of reduction of time, costs and energy consumption in road construction. Full scale demonstration projects are envisaged to be carried out during and after completion of the project, in order to validate and to implement the research results in four different European climates and economies in the following countries: Cyprus, Romania, Turkey and United Kingdom. The project intends to overcome the inherent scientific and technological barriers in steel tire-cord fibre processing, concrete technology and pavement design in order to be able , finally to deliver new processes, to develop specific life-cycle assessment models and comprehensive design guidelines. This paper presents the specific features of structural design of Long Lasting Rigid Pavements – LLRP for these demonstration projects.

2. DEMONSTRATION PROJECTS

2.1 Demonstration project in Eastern European Environment : Iasi, ROMANIA

2.1.1. Selection of demonstration project

The Romanian demonstration project will be carried out by the Regional Roads and Bridges Directorate, in its quality of part of Consortium and end user.

The road network of Regional Roads and Bridges Directorate of Iasi, as shown in Fig. 1, has a length of 3392 km and comprises 791 bridges[2].

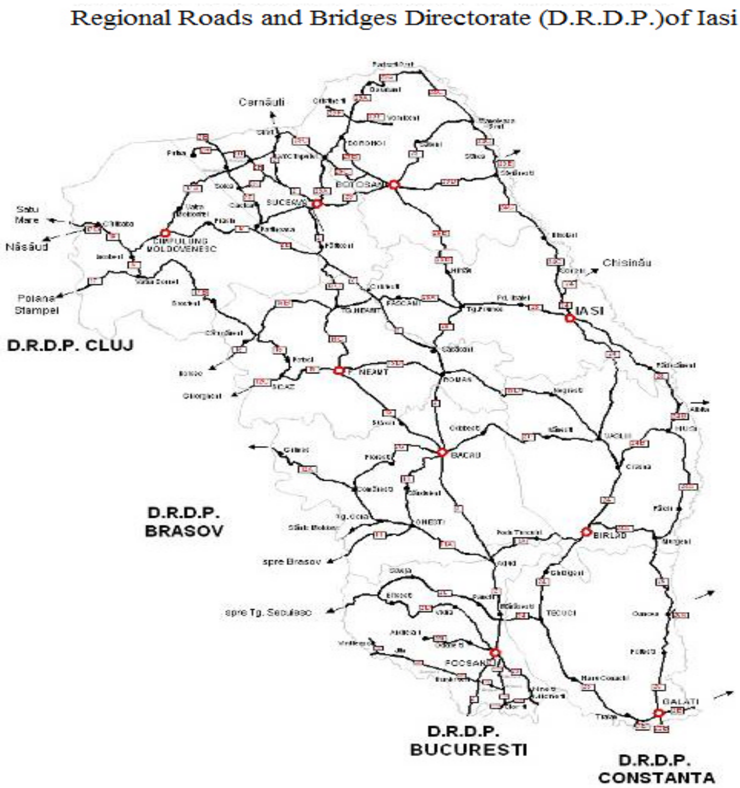


Fig.1. Schematic map of road network of Regional Roads and Bridges Directorate of Iasi [2]

Since 1998, Regional Roads and Bridges Directorate of Iasi has been responsible for rehabilitation of over 320 kilometers of their network. In this context part of National Road DN17 (E 576) Suceava – Vatra Dornei, namely the road sector from Km 217 to 218 (Fig 2) which is under rehabilitation process, has been selected as a demonstration project on the bases of the EcoLanes requirements.

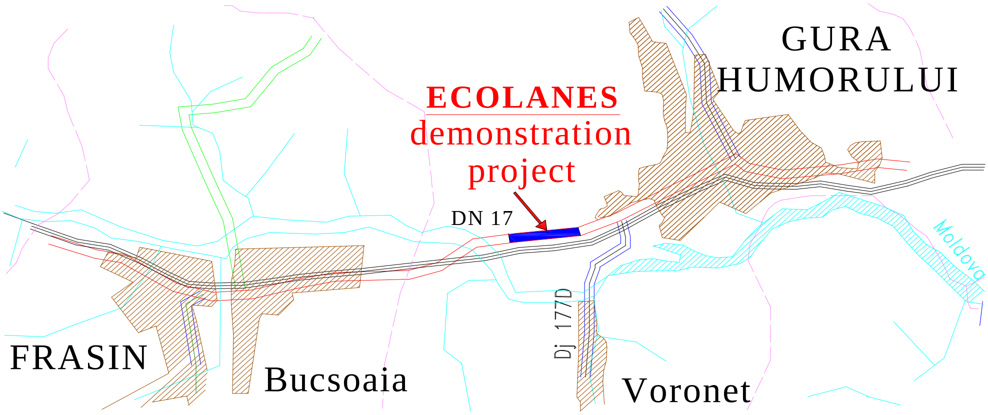


Fig.2 Location of the demonstration sector of National Road 17 [2]

From geological point of view, in this area, cohesive soils, such as silty and sandy clays and the local slopes consists the subgrade is considerate from other sedimentary and metamorphic rocks. The climatic conditions on the area include severe winters (-25^{0}C)and hot summers (+30^{o}C)and by high level of seasonal precipitation, depth of frost varying between 100and 110 cm.

Fig.3 General view of the existing road sector selected for demonstration project[2]

2.1.2. Aspects of structural design for the Romanian EcoLanes demonstration project

The design study has been conducted in concordance with the recommendations of the Romanian Standard NP 081 – 2002 [3], the main stages of the rigid pavement design being as follows:

- Establishing the design traffic;
- Establishing the bearing capacity of the foundation soil;
- Conceiving the rigid pavement structure;
- Establishing the bearing capacity at the level of the base course;
- Determining the thickness of the surface course.

2.1.2.1 Establishing the design traffic

The traffic calculation has been done using the standard recommendations. Preliminary studies have been undertaken in order to obtain relevant data concerning the composition, intensity and evolution of traffic, and also for the geotechnical characteristics of the foundation soil and the hydrologic regime on the site in accordance with the distribution of the climatic types (Fig. 4).

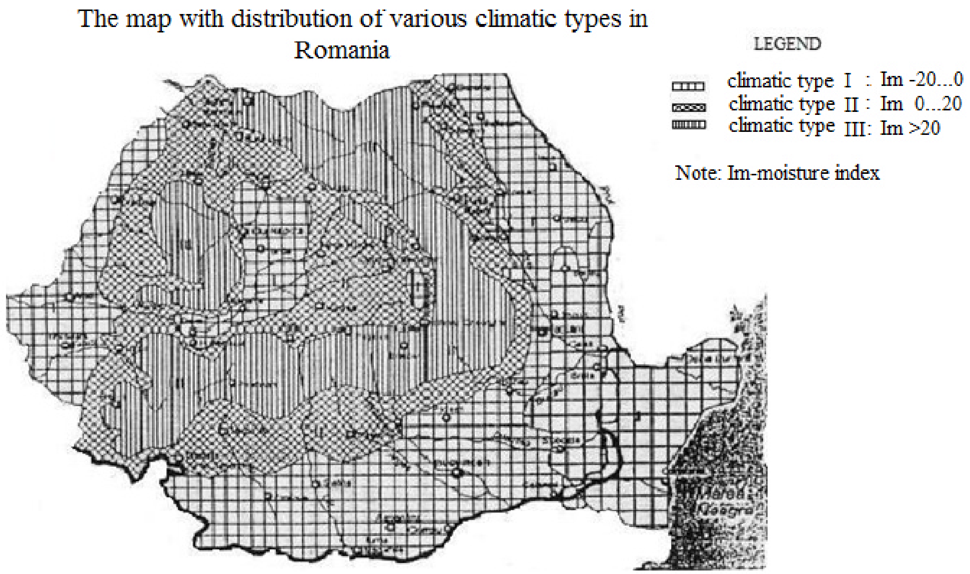


Fig. 4. Distribution of climatic types in Romania [3]

The design traffic, N_c, expressed in millions of standard axles (m.o.s.) of 115 KN, is established based on the average annual daily traffic (AADT) presented in Table 1, for a perspective period, p_p of 30 years and a coefficient transversal distribution for the lanes c_{rt} of 0,50.

Table 1. Traffic data for the demonstration project

Vehicle type	MZA(AADT) (millions standard axles)	pk - coefficient of evolution (Growth factor)	fek - equivalence coefficient(ESAL factor)	$MZA_k \cdot p_k \cdot f_{ek}$
2 axle trucks	358	2.68	0.30	288
3 and 4 axle trucks	224	1.83	3.80	1558
Articulated vehicle	296	1.74	2.90	1494
Buses	81	2.30	1.50	279
Farm tractors	11	2.04	0.20	4
Road trains	46	1.48	1.60	109
Total o.s. = $\sum_{k=1}^{6} MZA_k \cdot p_k \cdot f_{ek}$ =				3732

According the standard provisions the design traffic is calculated with the relation (1) as follows:

$$N_c = 365 \cdot 10^{-6} \cdot p_p \cdot c_{rt} \cdot \sum_{k=1}^{6} MZA_k \cdot p_k \cdot f_{ek} \qquad (m.o.s) \qquad (1)$$

Nc = 365 x 10^{-6} x 30 x 0.50 x 3732 = 20.43 m.s.a.

In accordance with the previous studies [4] for the demonstration project it was envisaged to use the following pavement of structures.

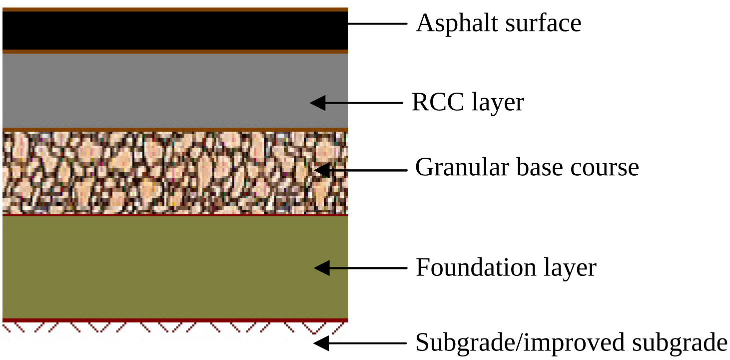


Fig .5 Option C1 - Asphalt surface course on RCC base and granular subbase [4]

For this type of pavement structure the thickness of the concrete slab has been determined performing the following steps:

2.1.2.2. Establishing the bearing capacity of the foundation soil

The coefficient of subgrade reaction, $K_{0,}$ is determined function of the climate, hydrologic regime, and type of soil, as given in the table below:

Table 2: Coefficient of subgrade reaction values, **K_0 [3]**

<table>
<tr><th rowspan="2">Climate Type</th><th rowspan="2">Hydrologic Regime</th><th colspan="5">Soil Type</th></tr>
<tr><th>P1</th><th>P2</th><th>P3</th><th>P4</th><th>P5</th></tr>
<tr><td rowspan="3">I</td><td>1</td><td rowspan="9">56</td><td rowspan="5">53</td><td>46</td><td rowspan="2">50</td><td>50</td></tr>
<tr><td>2a</td><td rowspan="5">44</td><td>48</td></tr>
<tr><td>2b</td><td>46</td><td>46</td></tr>
<tr><td rowspan="3">II</td><td>1</td><td rowspan="2">50</td><td>50</td></tr>
<tr><td>2a</td><td rowspan="2">46</td></tr>
<tr><td>2b</td><td>50</td><td>46</td></tr>
<tr><td rowspan="3">III</td><td>1</td><td>53</td><td rowspan="3">42</td><td>39</td><td>50</td></tr>
<tr><td>2a</td><td rowspan="2">50</td><td rowspan="2">37</td><td rowspan="2">44</td></tr>
<tr><td>2b</td></tr>
</table>

Note : The hydrological regime is distributed as follows:

- hydrological regime 1, corresponding to the FAVORABLE conditions , according STAS1709 / 2;
- hydrological regime 2, corresponding to the MEDIUM and UNFAVORABLE hydrological conditions, according the Romanian STAS 1709 / 2, as follows :
  - **2 a**: for embankment road sectors, with minimum height of 1.00 m;
  - **2 b**: for sectors of road located:
- in the mound with the height of beneath 1.00 m;
- at the ground height;
- at the mixed profile;
- in cutting.

In accordance with Tab. 2 based on the following parameters: type of soil: P3; climatic type: III and hydrological regime 2b, the value of the coefficient of sub-grade reaction is **K_0=42 MN/m^3.**

2.1.2.3. Conceiving the rigid pavement structure

The type of the rigid pavement structure selected is presented in Fig. 6 where a ballast foundation layer of 30 cm thicknese has been adopted according Standard recommendations.

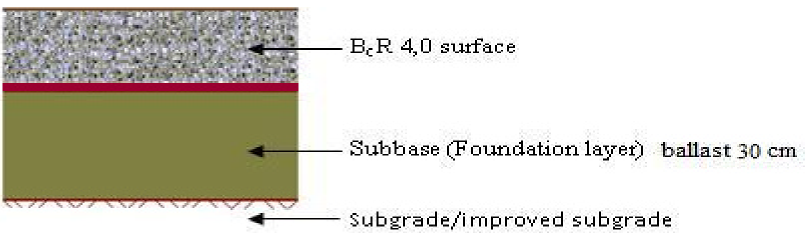


Fig. 6 The pavement structure selected for the Romanian demonstration project

2.1.2.4. Establishing the bearing capacity at the level of the sub-base

The bearing capacity at the level of the subbase, expressed by the coefficient of reaction at the surface of the subbase K, is determined function of the follow parameters:

- the coefficient of subgrade reaction K_0;
- the equivalent thickness of the base/sub-base courses, H_{ech}, representing the sum of the equivalent thicknesses of these layers, given by the following relationship:

$$H_{ech} = \sum_{i=1}^{n} h_i \times a_i \qquad (cm) \tag{2}$$

where:

n – number of layers;

h_i – the actual thickness of the layer “i”, in cm;

a_i – equivalence coefficient for the layer “i”

$$\mathrm{H}_{\mathrm{ech}} = 25 \cdot 0,75 = 18,75 \;\; cm$$

For the soil type $\mathbf{P_3}$, **climate type: III** and **hydrologic regime: 2b**, the coefficient of reaction at the surface of the subbase: **K=58 MN/m^3**.

2.1.2.5. Determining the thickness of the concrete slab

The design criterion is expressed as follows:

$$s \le s_{t,adm}$$

where:

s – the tensile stress from bending in the concrete slab, determined in various design hypothesis;

$s_{t,adm}$ – the allowable tensile stress from bending.

The allowable tensile stress from bending ($s_{t,adm}$) is determined by using the relationship:

s_{tadm}= $R^k_{inc.}$ x a x (0.70 – ? x log N_C) (3)

s_{tadm} = 5.0 x 1.1 x (0,70 – 0.05 x log 20.43)=3.48MPa

where:

R^k_{inc} – the characteristic bending strength of the concrete at 28 days;

a – the coefficient of the increasing concrete strength in the interval 28-90 days, equal to 1,1;

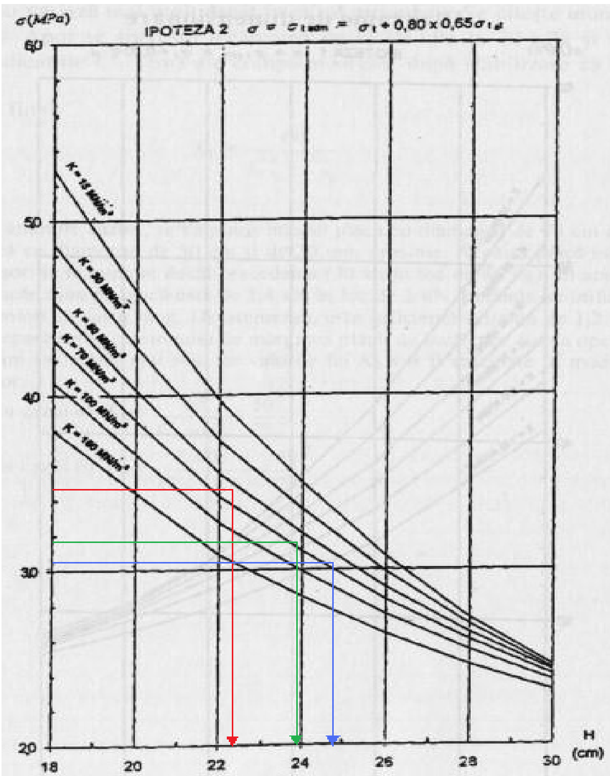
N_c – the traffic for the design period;

? – coefficient equal to 0.05;

The design hypotheses are:

1. $s = s_t + 0.8 \times s_{t\Delta t} \le s_{t,adm}$, for roads of technical class I and II roads;
2. $s = s_t + 0.8 \times 0.65 \times s_{t\Delta t} \le s_{t,adm}$, for roads of technical class III and IV roads;
3. $s = s_t \le s_{t,adm}$, for road of technical class V roads.

Finally in the design hypothesis number 2 and using the design diagram from Fig.7 for roads of technical class III, the modulus of subbase reaction K= 58 MN and the allowable flexural stress s_{tadm}=3,48 MPa a slab thickness Hsb=22cm has resulted.



Legend:

Slab thickness for Romania

Slab thickness for Cyprus

Slab thickness for Turkey

Fig. 7 Design diagram [3]

In order to avoid the direct contact of wheel loads with a surface of steel fibers reinforced concrete - SFRC slabs, it was decided to protect them with an asphalt layer of 8 cm thickness. For research reasons, in order to evaluate the performance of the Long Lasting Rigid Pavements with various thicknesses, three segnificant sectors are shown in Fig. 6, are envisaged[2] to be constructed in the frame of the demonstration project three sectors having the slab thickness of of 28 cm (sector no.1), 23 cm (sector no.2) and 18 cm (sector no. 3).

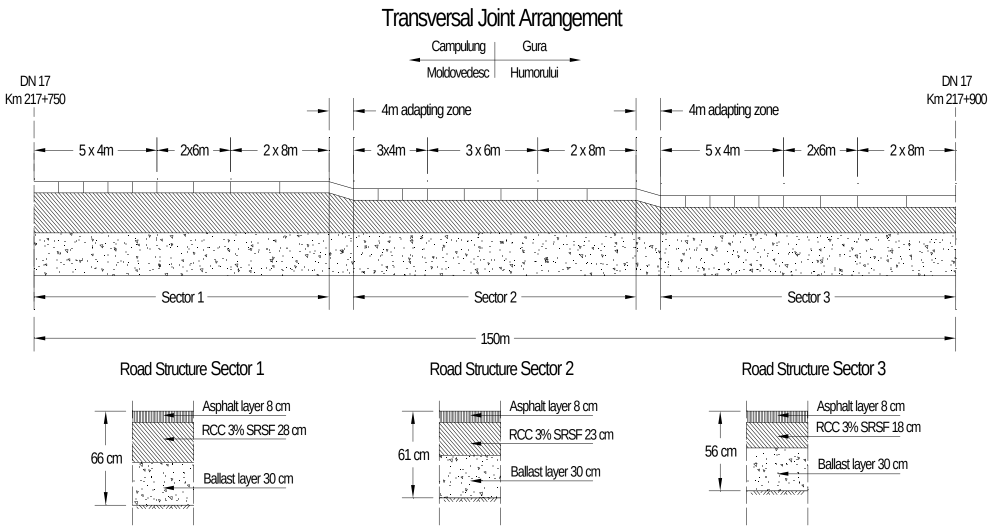


Fig.6. LLRP demonstration project with sectors with slabs of different thicknesses [2]

2.2. Demonstration project in Eastern Mediterrenean Environment: Paphos, CYPRUS

2.2.1. Selection of demonstration project

The demonstration project of Cyprus is envisaged to be carried out by the Public Works Department. According to 2002 statistics, the road network in the free areas of Cyprus as shown in Fig. 7 consists of about 7.206 km of paved and 4.387 km of unpaved roads [5].

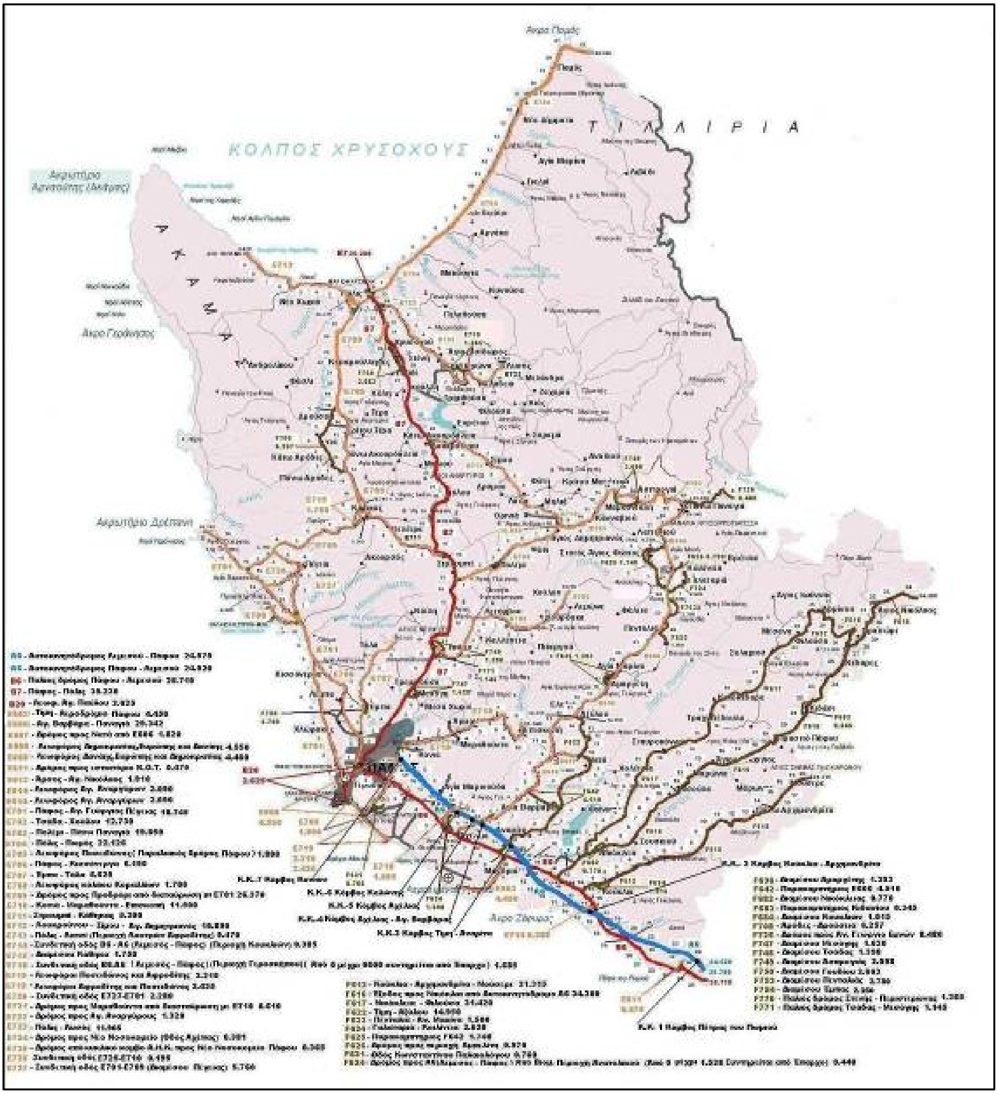


Fig.7. Pafos district's road network [5]

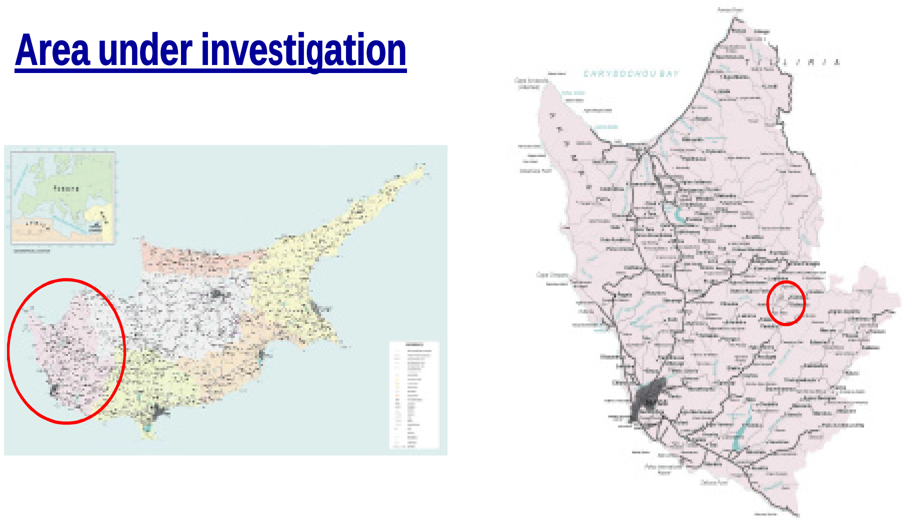


Fig. 8 Micro-Environment of the demonstration project [5]

The area where the demonstration project is situated is hilly to mountainous terrain with an altitude of 740 m above the sea level. Part of the road, the section (Km0+000 to 0+700) passes through the middle of a steep hillside (Fig. 9).

Fig.9 The section of the road sector choice for demonstration project[5]

In the past the area suffered from landslides, many of which are still active, the soil of the present geological formations being very disturbed. The high degree of deformation and alteration in combination with the underlying bentonitic clays and the steep topography is considered [5] as one of the main factors of instability in the Pafos region. In addition, the earthquakes have played (and still play) a very significant role.

2.2.2. Aspects of structural design for Cyprus EcoLanes demonstration project

2.2.2.1 Traffic data

The traffic calculation has been done based on the data made available by the Cyprus representative and using the recommendations of Romanian Standard NP 081–2002 “Technical recommendation for structural design of rigid road pavements”[3].

Table 3: Traffic data for the current design

Vehicle Type	MZA(AADT) (millions standard axles)	p_k - coefficient of evolution (Growth factor)	f_{ek} - equivalence coefficient(ESAL factor)	$MZA_k \cdot p_k \cdot f_{ek}$
2-axle trucks	40	2.68	0.3	32
3 or 4-axle trucks	25	1.83	3.8	174
Articulated vehicles	-	-	2.9	-
Buses	12	2.30	1.5	41
Tractors with/without trailer	-	-	0.2	-
Trains	-	-	1.6	-
Total o.s. 115= $\sum_{k=1}^{6} MZA_k \cdot p_k \cdot f_{ek}$ =				247

The input data are defined accordingly, and the design traffic has been established as follows:

N_c=365 x 10^{-6} x 30 x 0.5 x 247=1.35 (m.o.s.)

2.2.2.2. Determining the thickness of the sub-base course.

According to standard recommendation [3], the minimum thickness of the subbase layer must be of 25 cm (Fig 10).

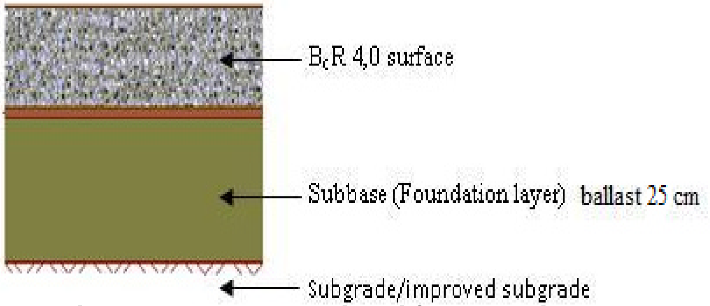


Fig. 10. The pavement structure selected for Cyprus demonstration project

Proceeding in a way similar with that used for the Romanian demonstration project, for the following input parameters: the coefficient of subgrade reaction

K_0=46 (soil type P_5, climate type II and hydrologic regime 2b), and H_{ech} = 18.75 a modulus of subbase **K = 58 MN/ m^3** has been obtained.

The laboratory tests results undertaken in Cyprus reveal a flexural strength R^K_{inc} = 4.0 Mpa. The allowable tensile stress from bending for the SFRC slab ($s_{t,adm}$) resulted as follows:

s_{tadm}= 4.0 x 1.1 x (0,70 – 0.05 x log 1.35)=3.05MPa

In the design hypothesis number 2 design diagram (Fig.7) for roads of technical class III introducing the modulus of subbase reaction K= 58 MN and the allowable flexural stress s_{tadm}=3,05 MPa a slab thickness Hsb=24cm has been obtained.

2.3. Demonstration project in Eastern Mediterranean Environment: Antalya TURKEY

In accordance with the existing statistics [6] the road network of Turkey as shown in Fig.11 has a total length of 63 156 Km includes the following categories of roads:

- Motorways : (2.9 %), State Highways: (49.7%), Local Highways : (47.4 %)

Fig. 11 The location of the demonstration project on the road network of Turkey

2.3.1. Selection of demonstration project

In accordance with fig.12 the demonstration project has been selected in the city of Antalya on the Necip Fazil street between Km 1+215and Km 1+800 The Antalya climate which is located 40m higher than the sea level, is a Mediterranean one characterized by hot and dry in summers, mild and rainy in winters. The average temperature in summer is 43,8 °C and the humidity rate is 50 %, the highest recorded temperature was 65,0 °C and the lowest 0 °C .

A general view of a selected demonstration project is shown in Fig.13.

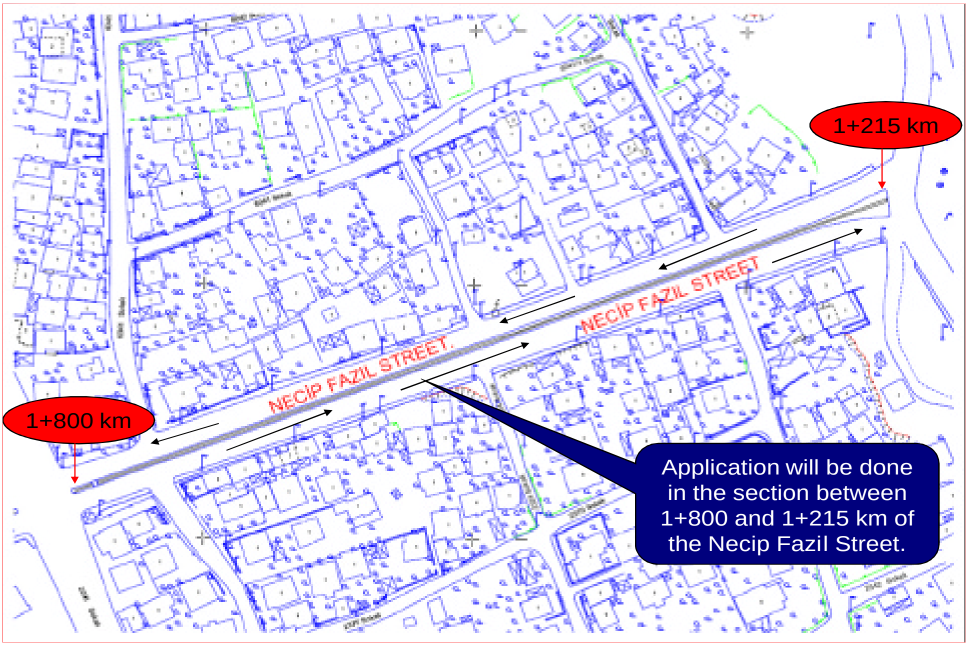


Fig. 12 Location of the demonstration sector [6]

Fig. 13 View of chosen demonstration sector [6]

2.3.2. Aspects of structural design for Turkey EcoLanes demonstration project

2.3.2.1 Traffic data

The traffic calculation has been done based on the data provided by the Turkish representation and by using the recommendations of Romanian Standard NP 081–2002 "Technical recommendation for structural design of rigid road pavements"[3],[7].

Table 4: Traffic data for the current design

Vehicle Type	MZA(AADT) (millions standard axles)	p_k - coefficient of evolution (Growth factor)	f_{ek} - equivalence coefficient(ESAL factor)	$MZA_k \cdot p_k \cdot f_{ek}$
2-axle trucks	72	2.68	0.3	58
3 or 4-axle trucks	181	1.83	3.8	1259
Articulated vehicles	73	1.74	2.9	368
Buses	214	2.30	1.5	738
Tractors with/without trailer	36	2.04	0.2	15
Trains	123	1.48	1.6	291
Total o.s. 115= $\sum_{k=1}^{6} MZA_k \cdot p_k \cdot f_{ek}$ =				2728

The design traffic N_c related according recomendations resulted as follows:

N_c=365 x 10^{-6} x 30 x 0.5 x 2728=14.93 (m.o.s.)

2.3.2.2. Determining the thickness of the surface course.

According to the standard recommendations a subbase layer of 20 cm and a base course of 10 cm as shown in Fig 14.

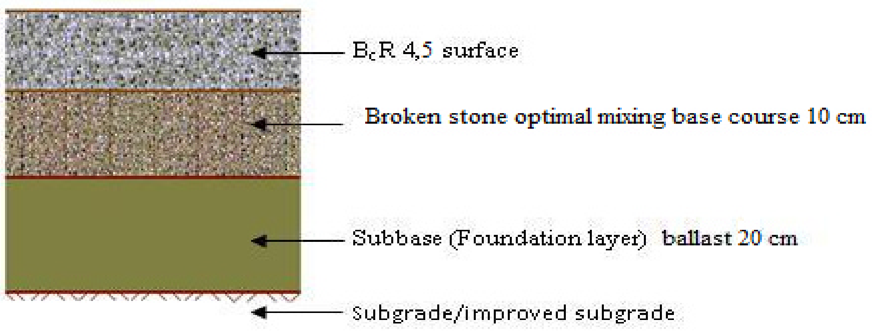


Fig. 14. The pavement structure selected for Turkey demonstration project

Proceeding in a similar way with that used for the Romanian demonstration project, for the input parameters: the coefficient of subgrade reaction K_0=46 (soil type P_5, climate type II and hydrologic regime 2b), and H_{ech} = 25, a modulus of subbase **K = 65 MN/ m³** has been obtained.
The laboratory tests results performed in Antalya laboratories revealed a flexural strength R^K_{inc} = 4.5 Mpa.

The allowable tensile stress from bending for the SFRC slab ($s_{t,adm}$) resulted as follows:

s_{tadm}= 4.5 x 1.1 x (0.70 – 0.05 x log 14.93)=3.17MPa

Finally in the design hypothesis number 2, by using the design diagram Fig.2 for roads of technical class III introducing the modulus of subbase reaction K= 65 MN and allowable flexural stress s_{tadm}=3,17 MPa,a slab thickness Hsb=23cm has been obtained.

3. CONCLUSIONS

Table 5 presents the synthetic results of the concrete slab thicknesses obtained for the three demonstration sectors investigated.

Table 5. Thickness of slabs, for various demonstration project

	Romania	Cyprus	Turkey
Design traffic (m.o.s)	20.43	1.35	14.93
Climate type	III	II	II
Modulus of subgrde reaction K_0	42	46	46
Modulus of subbase reaction K	58	58	65
Strength of the concrete at 28 days R^k_{inc}	5.0	4.0	4.5
Flexural strength s_{tadm}	3.48	3.05	3.17
Thickness of the concrete slab	22	24	23

On may concluded that despite of the variability observed in traffic and climatic conditions for varieus demonstration project, the thickness of the slabs are very similar, but their behaviors are expected to be different taking in to consideration the significant differences in environmental and traffic on.

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Landslide risk management during rehabilitation of transportation infrastructure

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Summary

The extensive rehabilitation works of transportation infrastructure are required by the increase of the traffic intensity in respect to speed and safety issues. Moreover, important damages are developed as consequences of local instability or a landslide activity in the zone of interest.

Landslide risk is defined by the vulnerability of the zone of interest multiplied with the landslide probability. The risk elements are continuously modified by the economic development of that zone.

Previous landslide activity recorded in the area of interest can be useful in terms of assessing the movement level and the activity of the rock mass. Although landslide probability is not assessed by accurate mapping of Romanian territory, more important should be the assessment of the vulnerability in the zones of high landslide potential that currently exists to large extent in most parts of the country.

Maps of the landslide potential can enter larger details when projects of road rehabilitation are developed in areas where the risk elements have increased significantly over the last decade.

When developing projects of road rehabilitation involving the increase of the traffic lanes, the landslide risk is increased and thus mitigation measures are necessary to consider. There are active measures and passive measures, and strongly orientated to protect the risk elements against life threat and property loss.

Monitoring systems can be organized as part of the risk management in order to set up alarm levels when recording displacements or water levels that generate the idea of increased landslide potential.

The paper presents management issues for risk assessment and mitigation solutions to reduce the landslide risk during road rehabilitation works.

KEYWORDS: risk management, landslide, road rehabilitation, mitigation solutions

1. INTRODUCTION

The cost-effective management of the natural disaster is a base concept for the long term development of all societies. Once the present problems related to landslides are defined as part of the natural disaster issue, new approaches and methods are needed that will simultaneously allow: the improvement of prognosis related to the place, time and characteristics of these natural phenomena, creating scenarios concerning the optimum strategies to adopt when such a disaster is triggered, as well as the adoption of post-disaster strategies in order to reduce the damages and re-install the normality within the community [1]. Elements that define such methods based on geo-sciences bring to attention a sum of experiences related to the field of landslide management with interventions on reducing the effects on society at large.

2. RISK ELEMENTS TO LANDSLIDES IN ROAD REHABILITATION

Various guidelines and research paper concluded that risk is general considered as a measure of the probability and vulnerability of an adverse effect to health, property or the environment. More accurately presented risk is defined as [2]:

- For life loss – the annual probability that the person most at risk will lose his/her life when considering the landslide hazard, the temporal and spatial probability and vulnerability of the person;
- For property loss – the annualized loss when considering the elements at risk, their temporal and spatial probability and vulnerability.

Elements at risk consist of the population, buildings and engineering works, social and economical activities, environmental features in the area potentially affected by the landslide hazard. As an example, the presence of a road in an area with a landslide hazard involves different elements at risk when compared to dwellings developed on a natural slope – figure 1 [3].

Once landslide risk has been analyzed and evaluated, the treatment component is naturally the following step to deal with the landslide event. In this respect, typical options would include:

- To accept the risk when risk is confined to acceptable domain;
- To avoid the risk when a new construction is in question in the potential area of a landslide risk, abandoning the project and looking for alternative construction sites where risk is acceptable;

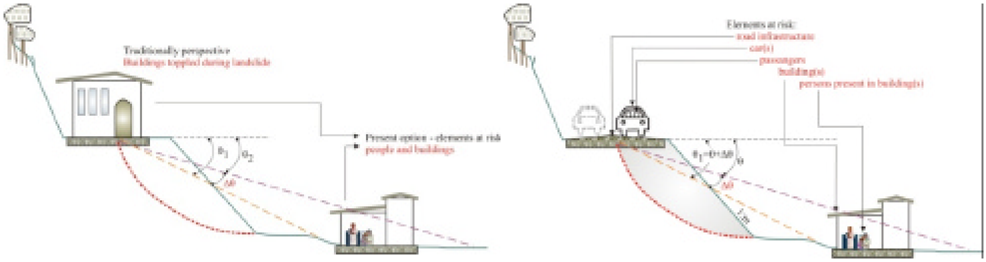

Figure 1. A different perspective of the landslide risk when various constructions change the nature of the elements at risk

- To reduce the likelihood that would require consolidation/stabilization measures to control triggering aspects; after implementation, the risk would fall within acceptable domain.
- To reduce consequences by setting defensive measures of stabilization.

3. CASE STUDY ON ROAD REHABILITATION

Rehabilitation design of roads relies on the site investigation results in order to accommodate both the pavement design with the re-assessed soil parameters and potential new retaining structures or bridges with the modified site conditions.

The rehabilitation design of the road sector on DN17 - 217+900km – 217+750km is part of the ECOLANES research project. The soil investigation performed on the site represented the first stage identification of the site conditions and is mainly referring to the existing soil characteristics under the present road structure. Disturbed and undisturbed soil samples have been removed, transported to the faculty laboratory of geotechnical engineering and subjected to laboratory tests to evaluate physical and mechanical soil parameters [4]. The concluded results of the investigation have been used in various design models proposed for the pavement design.

Economical and Sustainable Pavement Infrastructure for Surface Transport – ECOLANES is an FP6 STREP project with the aim of developing design and manufacture of new construction concepts for road infrastructure in order to meet the reduction of the construction cost and time by almost 15%, the energy consumption during construction by 40% and involve waste materials and minimize maintenance activities.

Roller compacted concrete – RCC involves as a construction material for rigid pavements steel fiber reinforced concrete that may alter the perspective of pavement design related to the joint interdistance between plates, the thicknesses of layers involved in the pavement structure and consequently meet the cost and time reductions aimed by the project.

One of the project stages refers to an experimental road sector performed according to the new design and technology established along the previous stages with RCC.

The sector is located in the North-Eastern part of Romania, on the National Road 17 between 217+900km – 217+750km (figure 2). The site conditions were investigated within the working group from the Technical University "Gheorghe Asachi", Faculty of Civil Engineering and Building Services, Iasi, Romania, assisted by the representatives from the Romanian National Road Authority.

Figure 2. Road sector conditions investigated for the rehabilitation project

The investigation points were set to accommodate both sides of the road given the fact that transversally, the existing road suggests a mixed profile - Figure 3.

The earth filling works have been performed at an inadequate compaction degree and thus, potential local landslides can be triggered under the road structure.

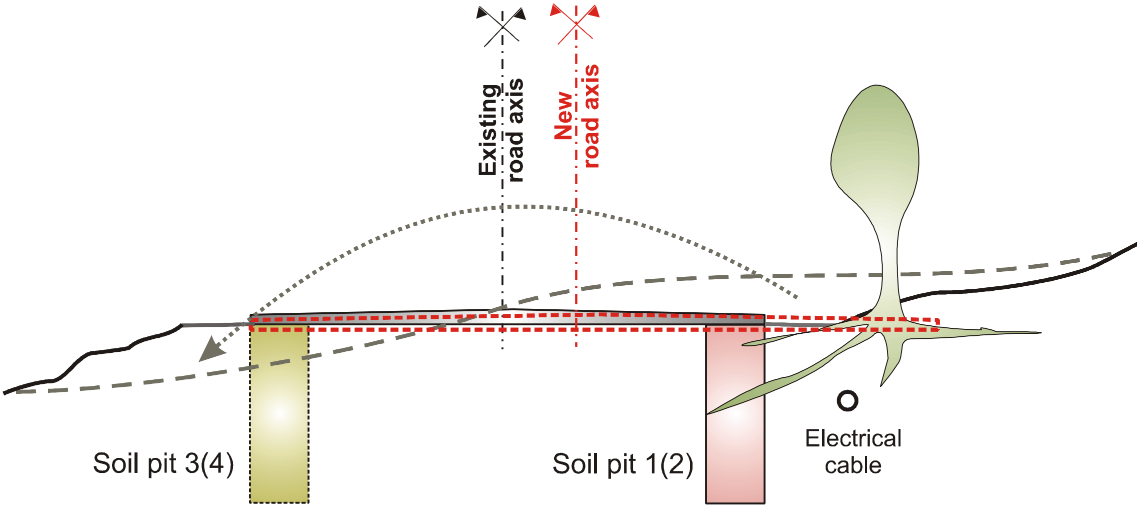

Figure 3. Cross section of the road sector in the initial site conditions

The road rehabilitation is intended to improve the traffic intensity and thus, a supplementary lane is added to the existing ones. The elements at risk are increased and the risk evaluation is modified by the change in the vulnerability for the new lane – Figure 4.

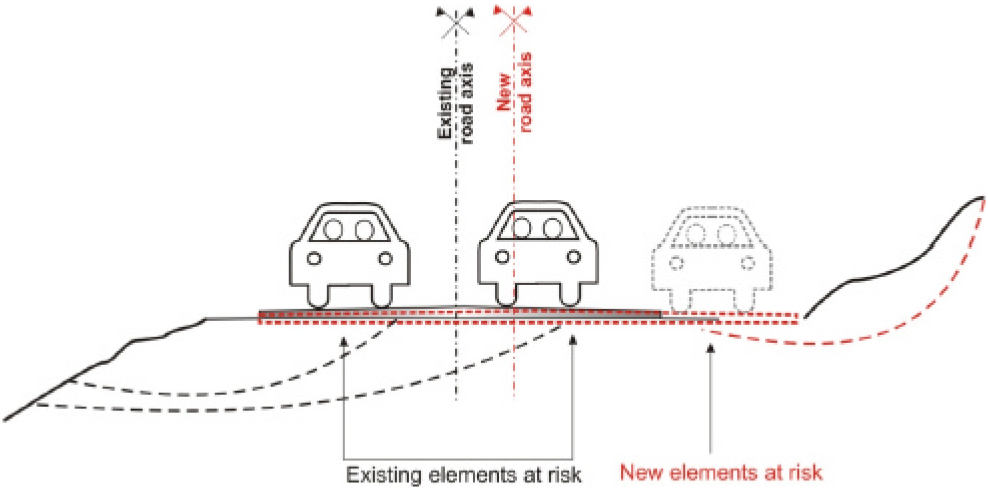

Figure 4. Risk evaluation based on different elements at risk after the road rehabilitation

Although cross sections may induce a clear perspective of the affected area, the top view of the developed soil movement is more effective to assess vulnerabilities and risks involved – figure 5.

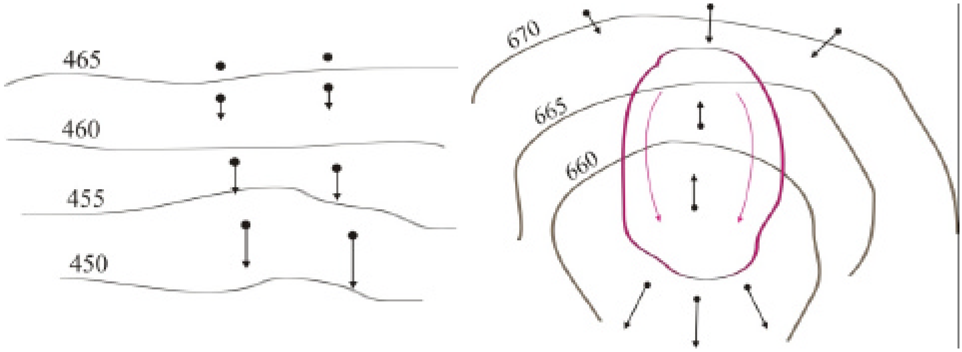

Figure 5. The assessment of the spatial development of the soil movement contributes to an accurate risk evaluation related to the road itself

The presented case is in a zone of low to medium landslide potential, but local landslides may occur up-hill the new constructed lane. Risk is non-existing for human loss but there is a risk as a property loss related to road structure and vehicles travelling in the area.

4. CONCLUSIONS

The national guides to create risk and hazard zoning related to landslides represent instruments that complete to a certain extent the necessary tools to implement the landslide control in Romania. Road rehabilitation projects increase the elements at risk. Mitigation measures to reduce the landsliding risk are considered based on the risk itself and thus, risk evaluation is very important and a national guide is the first necessary step in risk zoning.

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New additives for road bitumen

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Summary

The paper presents the results of laboratory tests for two additives ADIROL ALCAMID F and ADIROL ALCAMID FS. Three mixtures were made in laboratory, with two types of bitumen of 4 dosages each, in order to assess characteristic physical and mechanical values of mixtures made with bitumen having 0,5 % additives.

The analysis of results shows a substantial improvement of the characteristic physical and mechanical values of these mixtures as compared to mixtures made with non-additive bitumen.

KEYWORDS: bitumen, additives, aggregate.

1. INTRODUCTION

The use on a large scale of natural ballast-pit aggregates - which have a content of SiO_2 exceeding 65 % - in the composition of asphalt mixtures requires the fact that we should consider special measures on insuring the bitumen's adhesiveness to these acid aggregates. One of these measures is to aditivate bitumen with different tensioactive substances.

2. ADDITIVES TESTED IN THE LABORATORY

In order to study the influence of the following additives: ADIROL ALCAMID F and ADIROL ALCAMID FS over road bitumen, there have been made tests in the ROADS laboratory at the Technical University of Iasi, Faculty de Civil Engineering, on two types of bitumen: the first type manufactured by Astra refinery from Ploiesti and the second manufactured by Suplacu refinery from Barcau (Bihor-Romania).

Both types of bitumen have been determined the main characteristic values, both in pure state and in mixture with 0.5 % (of weight) with the two previously mentioned additives - everything is presented in the Table 1.

Table 1: Characteristics of both types of bitumen

Type of bitumen	Characteristics			
	Penetration at 25°C, 1/10 mm	Softening point (I.B.)	Penetration index (P. I.)	‘a’ susceptibility to heat
ASTRA PLOIESTI Refinery				
Witness bitumen	84	51,8	-0,615	0,037
Bitumen +0.5% additive F	91	49,6	+0,278	0,038
Bitumen +0.5% additive FS	76	51,2	-0,166	0,039
SUPLACU DE BARCAU Refinery				
Witness bitumen	103	47,2	-0,017	0,040
Bitumen +0.5% additive F	*101*	*48,1*	*+0,185*	*0,039*
Bitumen +0.5% additive FS	86	49,0	-0,059	0,040

Both pure state (witness) and aditivated types of bitumen have been used as binding material in the composition of three mixtures: an asphalting concrete rich in chippings - BA 8 and an asphalting concrete rich in chippings – BA 16, both with chippings of 4-8, 8-16 and crusher sand of 0-4 from Turcoaia (Tulcea) quarry: granite, and an asphalting concrete rich in chippings – BA 8 with chippings of 4-8 and crusher sand of 0-4 from Homorod (Harghita) quarry: andesite.

The granulometric curves of natural aggregates as well as the dosages used for each mixture as well as the granulometric curves of mixtures of aggregates are presented in the Table 2, and the values of the impurities from the natural sands are presented in the Table 3.

The granulometric curves of natural aggregates are framed in the SR 174-1/2002 (mixtures I and II) and the SR 174 -1/2002 (mixture III).

In the laboratory there have been made asphalting mixtures with 4 dosages of bitumen for each type of asphalting concrete out of which there have been produced 8 Marshall-type cylinder samples for each (D = 10.16 cm and H = 6.35 cm) for which there have been determined the values of the physical and mechanical characteristics presented in the Tables 4 and 5.

The variations of the values of physical and mechanical characteristics of experimental mixtures as comparing to witness mixtures are presented in the Table 6 for optimal dosages of bitumen.

Table 2. The composition of mixtures of natural aggregates and granulometric curves for mixtures I, II and III.

			Mixtures		
			I – BA 8	II – BA 16	III – BA 8
Passed (%) through:	Screen:	Ø 25	-	100,00	-
		Ø 16	-	95,59	-
		Ø 10	100,00	71,13	100,00
		Ø 8	89,16	61,60	93,38
	Sieve:	# 4	60,64	47,57	72,10
		# 2	48,37	39,50	53,63
		# 1	39,54	32,92	42,66
		# 0,63	34,47	29,15	36,38
		# 0,2	17,92	16,42	17,79
		# 0,1	11,97	10,82	1012
		# 0,071	9,34	9,13	7,90
Chippings 4-8 (Turcoaia)			45,00	20,00	-
Chippings 8-16 (Turcoaia)			20,00	35,00	-
Chippings 4-8 (Harghita)			-	-	45,00
Crusher sand 0-4 (Turcoaia)			-	15,00	-
Crusher sand 0-4 (Harghita)			-	-	22,00
Natural sand (Tecuci)			23,00	18,00	-
River sand (Timisesti)				-	25,00
Filler (Bicaz)			12,00	12,00	8,00

Table 3. Values of the content of impurities in the river sands, used for mixture composition

Natural aggregate	Characteristics	
	Part that can be levigated (%)	Content of humus (the colour of the solution of 3% NaOH)
Natural sand 0 ... 4 Tecuci	0,94	Light yellow
Natural sand 0 ... 4 Timisesti	1,95	Light yellow

Table 4. The values of physical and mechanical characteristics of mixtures made with bitumen ASTRA Ploiesti.

No. crt.	Mixture type	Bitumen type	Bitumen dosage (%)	?$_a$ (kg/m^3)	A $_{vol.}$ (%)	Marshall trial		
						S (kN)	I (mm)	S/I (kN/mm)
1	I-BA 8 WITNESS (Turcoaia)	ASTRA Ploiesti	5,00	2367	1,678	7,6	1,90	4,000
			5,25	2387	0,501	7,9	2,70	2,926
			5,50	2393	0,248	9,7	3,93	2,468
			5,75	2383	0,198	9,3	4,70	1,979
2	I-BA 8 F (Turcoaia)	ASTRA Ploiesti	5,00	2374	1,252	10,0	2,60	3,846
			5,25	2379	0,859	8,5	3,70	2,194
			5,50	2396	0,132	10,0	4,33	2,309
			5,75	2390	0,020	8,6	4,93	1,744
3	I-BA 8 FS (Turcoaia)	ASTRA Ploiesti	5,00	2368	1,544	8,8	3,55	2,479
			5,25	2381	0,718	10,5	4,20	2,500
			5,50	2394	0,126	10,6	4,40	2,409
			5,75	2392	0,093	8,4	5,48	1,533
4	II-BA16 WITNESS (Turcoaia)	ASTRA Ploiesti	4,25	2404	1,093	12,2	3,77	3,236
			4,50	2407	0,701	11,9	4,46	2,668
			4,75	2414	0,461	9,0	4,53	1,987
			5,00	2410	0,266	8,3	5,27	1,575
5	II-BA 16 F (Turcoaia)	ASTRA Ploiesti	4,25	2393	1,414	11,2	3,43	3,265
			4,50	2397	0,838	13,4	4,33	3,095
			4,75	2414	0,272	10,0	4,78	2,092
			5,00	2405	0,258	10,2	5,47	1,865
6	II-BA 16 FS (Turcoaia)	ASTRA Ploiesti	4,25	2392	1,223	9,2	2,45	3,755
			4,50	2407	0,423	10,0	3,17	3,155
			4,75	2413	0,310	9,6	4,20	2,286
			5,00	2410	0,288	10,2	4,33	2,356
7	III-BA 8 WITNESS (Harghita)	ASTRA Ploiesti	5,50	2349	2,993	9,1	2,20	4,136
			5,75	2356	2,583	8,7	2,43	3,580
			6,00	2362	1,768	10,4	3,43	3,032
			6,25	2355	1,420	9,0	3,93	2,290
8	III-BA 8 F (Harghita)	ASTRA Ploiesti	5,50	2324	3,918	8,3	3,13	2,652
			5,75	2347	2,936	8,8	3,20	2,750
			6,00	2353	2,267	8,1	3,35	2,418
			6,25	2351	1,723	10,5	4,13	2,542
9	III-BA 8 FS (Harghita)	ASTRA Ploiesti	5,50	2311	3,710	7,8	2,95	2,644
			5,75	2343	2,015	9,1	3,80	2,395
			6,00	2366	0,911	10,0	3,95	2,532
			6,25	2363	0,408	9,8	5,47	1,792

Table 4 (continuation). The values of physical and mechanical characteristics of mixtures made with bitumen ASTRA Ploiesti.

No. crt.	Mixture type	Swelling, (%) vol. after … days			
		7	14	21	28
1	I-BA 8	0,000	0,013	0,020	0,287
	WITNESS	0,000	0,079	0,013	0,191
	(Turcoaia)	0,000	0,112	0,000	0,092
		0,000	0,146	0,066	0,258
2	I-BA 8	0,000	0,000	0,039	0,170
	F	0,026	0,000	0,065	0,269
	(Turcoaia)	0,000	0,000	0,020	0,172
		0,000	0,000	0,033	0,147
3	I-BA 8	0,000	0,364	0,039	0,117
	FS	0,065	0,287	0,055	0,190
	(Turcoaia)	0,039	0,190	0,125	0,164
		0,106	0,178	0,160	0,099
4	II-BA16	0,107	0,000	0,047	0,000
	WITNESS	0,139	0,000	0,086	0,000
	(Turcoaia)	0,033	0,000	0,127	0,000
		0,109	0,040	0,047	0,000
5	II-BA 16	0,000	0,020	0,119	0,020
	F	0,000	0,000	0,000	0,000
	(Turcoaia)	0,000	0,106	0,000	0,046
		0,000	0,060	0,000	0,060
6	II-BA 16	0,000	0,000	0,000	0,067
	FS	0,000	0,000	0,000	0,119
	(Turcoaia)	0,106	0,099	0,086	0,205
		0,027	0,000	0,073	0,147
7	III-BA 8	0,000	0,766	0,806	1,254
	WITNESS	0,098	0,540	0,628	0,903
	(Harghita)	0,000	0,204	0,145	0,394
		0,040	0,337	0,516	0,621
8	III-BA 8	0,060	0,381	0,689	1,162
	F	0,072	0,287	0,267	0,847
	(Harghita)	0,109	0,424	0,128	0,700
		0,013	0,033	0,000	0,190
9	III-BA 8	0,000	0,489	1,096	1,434
	FS	0,000	0,210	0,413	0,597
	(Harghita)	0,000	0,032	0,221	0,739
		0,000	0,000	0,007	0,039

Tabelul 5. The values of physical and mechanical characteristics of mixtures made with bitumen SUPLACU DE BARCAU - Bihor

No. crt.	Mixture type	Bitumen type	Bitumen dosage (%)	?$_a$ (kg/m^3)	A $_{vol.}$ (%)	Marshall trial		
						S (kN)	I (mm)	S/I (kN/mm)
1	I-BA 8	Suplacu	5,00	2377	1,116	9,3	3,07	3,029
	WITNESS	de	5,25	2378	0,769	8,4	3,83	2,193
	Turcoaia	Barcau	5,50	2386	0,248	8,3	4,23	1,962
			5,75	2380	0,239	8,8	4,80	1,833
2	I-BA 8	Suplacu	5,00	2378	0,921	9,5	2,67	3,558
	F	de	5,25	2384	0,667	8,7	3,60	2,417
	Turcoaia	Barcau	5,50	2398	0,205	9,2	3,70	2,486
			5,75	2396	0,027	8,3	4,75	1,747
3	I-BA 8	Suplacu	5,00	2389	0,848	10,2	3,17	3,218
	FS	de	5,25	2391	0,604	8,8	3,40	2,588
	Turcoaia	Barcau	5,50	2405	0,013	8,0	4,07	1,966
			5,75	2397	0,000	7,4	4,33	1,709
4	II-BA 16	Suplacu	4,25	2385	1,663	10,8	3,03	3,564
	WITNESS	de	4,50	2400	0,763	10,3	4,25	2,424
	Turcoaia	Barcau	4,75	2413	0,318	10,7	4,60	2,326
			5,00	2411	0,139	10,2	5,38	1,896
5	II-BA 16	Suplacu	4,25	2388	1,272	11,4	3,00	3,800
	F	de	4,50	2408	0,486	9,2	3,55	2,592
	Turcoaia	Barcau	4,75	2411	0,304	8,4	4,00	2,100
			5,00	2407	0,192	9,5	4,50	2,111
6	II-BA 16	Suplacu	4,25	2397	1,058	12,3	2,60	4,731
	FS	de	4,50	2408	0,766	9,8	2,80	3,500
	Turcoaia	Barcau	4,75	2410	0,424	9,4	3,42	2,749
			5,00	2404	0,265	9,6	4,00	2,400
7	III-BA 8	Suplacu	5,50	2339	3,346	10,5	2,27	4,626
	WITNESS	de	5,75	2362	2,225	9,7	2,60	3,731
	Harghita	Barcau	6,00	2368	0,692	9,8	2,93	3,345
			6,25	2364	0,297	8,1	3,90	2,077
8	III-BA 8	Suplacu	5,50	2335	2,654	8,3	1,85	4,486
	F	de	5,75	2337	2,033	7,8	2,48	3,145
	Harghita	Barcau	6,00	2342	1,698	8,2	3,07	2,671
			6,25	2335	1,458	8,8	3,80	2,316
9	III-BA 8	Suplacu	5,50	2348	2,667	9,7	2,73	3,552
	FS	de	5,75	2363	2,363	10,6	3,47	3,055
	Harghita	Barcau	6,00	2373	0,669	11,9	3,83	3,107
			6,25	2367	0,573	10,0	4,36	2,294

Tabelul 5 (continuation. The values of physical and mechanical characteristics of mixtures made with bitumen SUPLACU DE BARCAU – Bihor (continue)

No. crt.	Mixture type	Swelling, (%) vol. after … days			
		7	14	21	28
1	I-BA 8	0,000	0,069	0,000	0,000
	WITNESS	0,000	0,000	0,000	0,000
	Turcoaia	0,000	0,000	0,000	0,000
		0,000	0,000	0,000	0,000
2	I-BA 8	0,000	0,000	0,000	0,007
	F	0,000	0,000	0,000	0,013
	Turcoaia	0,000	0,000	0,000	0,000
		0,000	0,027	0,000	0,027
3	I-BA 8	0,000	0,013	0,000	0,000
	FS	0,000	0,000	0,000	0,039
	Turcoaia	0,000	0,093	0,086	0,000
		0,000	0,066	0,013	0,000
4	II-BA 16	0,000	0,000	0,000	0,000
	WITNESS	0,000	0,000	0,000	0,000
	Turcoaia	0,000	0,007	0,000	0,000
		0,000	0,000	0,007	0,000
5	II-BA 16	0,000	0,000	0,000	0,000
	F	0,000	0,000	0,000	0,000
	Turcoaia	0,000	0,000	0,000	0,000
		0,000	0,000	0,000	0,000
6	II-BA 16	0,000	0,010	0,100	0,040
	FS	0,000	0,000	0,066	0,000
	Turcoaia	0,000	0,000	0,000	0,000
		0,000	0,000	0,046	0,000
7	III-BA 8	0,000	0,171	0,020	0,237
	WITNESS	0,000	0,118	0,033	0,138
	Harghita	0,000	0,000	0,000	0,000
		0,000	0,039	0,000	0,000
8	III-BA 8	0,000	0,000	0,000	0,000
	F	0,000	0,000	0,000	0,000
	Harghita	0,000	0,000	0,000	0,000
		0,000	0,000	0,000	0,000
9	III-BA 8	0,484	0,729	0,749	1,108
	FS	0,231	0,442	0,336	0,501
	Harghita	0,000	0,007	0,000	0,013
		0,007	0,027	0,007	0,066

Table 6. Variation of physical and mechanical values of experimental mixtures as comparing to witness mixtures (optimal dosages).

Bitumen type	Mixture type	Optimal dosage (%)	Additive	Apparent density ?$_a$ (%)	Water absorption vol (%)	Marshall Stability (S) (%)	Running Index (I) (%)
ASTRA PLOIESTI	BA 8 (I)	5,50	F	100,13	53,23	103,09	110,18
	BA 8 (III)	6,00	F	99,62	128,22	77,88	97,67
	BA 16 (II)	4,75	F	100,00	59,06	111,11	105,52
	BA 8 (I)	5,50	FS	100,04	50,81	109,28	111,96
	BA 8 (III)	6,00	FS	100,17	51,53	96,15	115,16
	BA 16 (II)	4,75	FS	99,96	67,25	106,67	92,72
SUPLACU DE BARCAU	BA 8 (I)	5,50	F	100,50	82,66	110,84	87,47
	BA 8 (III)	6,00	F	98,90	245,38	83,67	104,78
	BA 16 (II)	4,75	F	99,92	95,60	78,50	86,96
	BA 8 (I)	5,50	FS	100,80	50,24	96,39	96,22

3. CONCLUSIONS

From the analysis of the results obtained after the research there has been found that:

1. Using the additives does neither remarkably modify the values of the main characteristics of the bitumen (penetration at + 25^0 C and softening point) and neither the initial type of structure (sol-gel) characteristic to the road bitumen;
2. All the physical and mechanical characteristics of the experimental mixtures suffer net improvements with the exception of the apparent density whose value remains practically unchanged;
3. For the experimental mixtures made using optimal bitumen proportions there has been remarked that:

- the water absorption reduces, with three exceptions, with 38 % on the average comparing to the witness mixtures; even in the case of the three exceptions the values of the water absorption are much below the maximum admitted limit - 5 % - of SR 174-1/2002: table 13 (ANEXA I);

- both the values of the stability and of the Marshall running index are practically constant, the variations being reduced: (- 22 % .... + 21 %) to the Marshall stability and (- 26 % ... + 31 %) to the Marshall running index;
- although determining the swelling values is not compulsory according to the latest standard SR l74-1/2002 there have been determined the swelling values at: 7, 14, 21 and 28 days. As it can be observed in the tables 4 and 5, the values obtained are very low (under 0.8 %) signaling a good behavior of the mixtures in time to water corrosion.

Considering the facts we have proved so far, we can state that using the additives ADIROL ALCAMID (Variants: F and FS) the characteristics of the road bitumen are not practically modified, but significantly diminish the values of water absorption to the mixtures made with natural acid aggregate (granite de Turcoaia) to which the road un aditivated bitumen have a reduced adhesiveness.

That creates the premises of a good behavior in time of asphalt mixtures and, implicitly, of an improved resistance to freezing and de-freezing, which insures the improvement of the road pavement viability realized with these mixtures in the wearing layer.

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Development of A Road Base Test Kit

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Introduction

Low volume rural roads comprise about 700,000 km of Australia’s 1 million km road network, providing access to many local communities, for tourists and contribute to the economic viability of the rural sector. They are predominately two-lane two-way roads, most are unsealed while the remainder generally have a spray sealed surface.

Obtaining economic and suitable road pavement materials for the effective construction and maintenance of the road network, particularly in rural and remote areas is an ongoing challenge for local practitioners. Increasing use of readily available local materials, often of marginal quality, is required to enhance the efficiency and effectiveness of the construction and maintenance of low volume roads. This problem is not unique to Australia.

Local materials generally comprise weathered rocks, soft rocks, ridge gravels, stream gravels and sands, sometimes mixed with clays, which are close to the construction site and can usually be won and placed by readily-available construction equipment. Some types of local material (e.g. sandstone, siltstone, calcrete and laterite) may require some degree of processing (e.g. grid-rolling, coarse screening over a grizzly or primary crushing). Other possible sources are by-products from local industry such as quarry spoil, slag and ash etc.. Many such materials have been used across Australia (Giummarra 2000, Robinson et al 1999) where local knowledge and experience have demonstrated satisfactory performance. Where a new material source is to be located and evaluated, and where testing resources and expertise are expensive and/or difficult to access, then there is need for an inexpensive and readily available means to assess any new material.

The purpose of this project, funded by DOTARS[1], is to develop a simple and inexpensive means of conducting at least an initial evaluation of new road base source materials. The objective is to produce a man portable road base test kit suitable for use by non-professional employees in Australia’s rural areas. The

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[1] Commonwealth of Australia Department of Infrastructure, Transport and Regional Development and Local Government.

potential for application in other countries is obvious with appropriate adaptation of the equipment and software to meet local conditions.

1.THE PROJECT PROCESS

This paper outlines the initial stages of the development of the **ARRB Road Base Test Kit** for such material evaluation. It describes an approach to material selection based on a few simple tests suitable for direct use in the field by local supervisory technical staff, supplemented by assessment of the results against simple criteria based on, but modified from, typical 'standard' specifications.

The process has been to assemble simple testing equipment for the determination of particle size grading and plasticity of potential pavement base course materials. This equipment package is being trialed by small workshops with local government technical staff using local materials to determine the most suitable equipment, to establish testing protocols, based on but simplified from Australian Standard procedures and, to develop a software package to facilitate an evaluation of the material properties.

The data collected during these trials are being compared with standard laboratory tests of the same materials to validate the simplifications. A caution must be sounded at this point. The Road Base Test Kit together with the procedures, calculations and estimates are not equivalent to a full scale professional testing laboratory evaluation. However, in the absence of such an evaluation the Road Base Test Kit will provide a first estimate on which to make decisions as to the use of available materials.

2. MATERIAL REQUIREMENTS

Unsealed low traffic road pavements have been constructed using an extremely wide variety of materials. The basic material grain size distribution requirements are succinctly described by Wooltorton (1947).

Material criteria are the gradation, or particle size distribution, and the plasticity of the fine material. Estimates of the maximum compacted density and optimum moisture content parameters may then be made from the results of these simple tests (Berney, E.S. & Wahl, R.S. 2007). In many cases the usual State Road Authorities materials requirements can be relaxed for low traffic routes where the risk of lower performance can be offset against costs of using better materials.

Relaxed standards are directly related to local conditions and it is not possible to establish valid nation-wide requirements as many 'non-standard' materials have been successfully used. It is suggested therefore that three categories of low-volume pavement types be considered;

- Bituminous sealed pavements; where the materials should be closely similar to the appropriate requirements established by the State authority.
- Unsealed pavements with a wearing course; where the wearing course material will have a higher fines content than the base to provide a smoother ride and reduce ravelling under dry conditions
- Unsealed pavements without a wearing course; where the base course also serves as the wearing course.

Guidelines for the specification of these three types of pavement are being developed and a typical example is given in Table 1.

As the range of gradation and plasticity properties commonly specified is very large the following guidelines are suggested for general use for low volume road pavements. Note, these requirements may be modified in the light of local experience.

Table 1 Guidelines for natural gravel material properties for unsealed low volume road pavement base course (adapted from Mulholland 1989)

Sieve Size (mm)	**Max Size 20mm** **Percent passing**
37.5	
26.5	100
19	93-100
9.5	71-87
4.75	47-70
2.36	35-56
0.425	14-32
0.075	6-20

Plasticity Index	Less than 500 mm annual rainfall max 20 - More than 500 mm annual rainfall max 12 OR (Use the Binder Index viz PI x % passing 0.075) max 500 for low rainfall and max 250 for > high rainfall
Soaked CBR	Minimum 50%

3. THE ROAD BASE TEST KIT EQUIPMENT

The kit (Plates 1 and 2) contains test equipment to perform the following activities:

- Sample preparation,
- Grading: 7 sieves,(37.5, 19, 9,5, 4.75, 2.36, 0.425 and 0.075 mm),
- Liquid Limit Device,

- Linear Shrinkage
- ARRB software-to perform all necessary calculations and to archive data
- A robust and lockable case to transport, store and protect the equipment.

3.1 Gradation

In assessing any material the first property to be determined is the gradation or particle size distribution. A gradation close to the theoretical maximum density gradation is clearly desirable. The sample gradation is determined by the simple sieve test procedure.

3.2 Plasticity

The plasticity of a pavement material is directly related to the presence of a 'fines' content, that is, particles smaller than 0.425 mm. Plasticity is defined by three parameters, the Liquid Limit (LL), above which the soil behaves almost as a liquid, the Plastic Limit (PL) above which the soil behaves like a plastic solid and can be readily moulded, and the Linear Shrinkage (LS) the likely amount a soil will shrink. A further parameter, the Plasticity Index (Ip), defined as the Liquid Limit minus the Plastic Limit (Ip = LL – PL) is frequently used to indicate the amount and activity of the fines/clay content of a material and is often restricted to a maximum value in specifications for pavement materials. The test kit gives simple procedures for determining LL and LS and approximates Ip from LS.

3.3 Hand calculations and spreadsheet

The results sheet (Appendix A) records values, determined on site using the test kit procedures, to be input directly into a spreadsheet calculation program on the ARRB web site (www.arrb.com.au) which will calculate the required material parameters, grain size distribution and plasticity. The software will also estimate the maximum modified compacted dry density and moisture content using equations under development by ARRB.

A specific and important feature is that using the ARRB web site to calculate the parameters will also automatically archive the data at ARRB for future use to refine the predictive equations.

4. INTERPRETATION

The results directly measured by the test kit, Table 2, give material grading and plasticity data, together with a classification of stone quality and a measure of soil water salinity, which can be assessed against criteria provided by the user or established as default values in the software.

5. ROAD BASE TEST KIT SOFTWARE

The spreadsheets provided with the test kit (appendix A) process raw test data to yield grading and plasticity figures for comparison with the above guidelines or any others adopted, Table 2. The spreadsheet includes provision for the user to input specific grading and plasticity requirements. Where a material does not meet the requirements then the software will carry out mix design calculations using up to three source materials.

The software will also plot the grading curve(s) for tested material(s), design a mix to meet a required grading and plot the resulting design mix against that requirement. It will also plot the mix on the Wooltorton (1947) and Jones and Paige-Green (1996) charts to allow consideration of the likely behaviour of the material.

Table 2 Use of the Test Kit

Input	Output	Predictions
Grading (particle size distribution) from 37.5 to 0.075 mm	Particle size distribution, comparison with user of default specifications	Compacted density, optimum moisture content, California Bearing Ratio.
Plasticity (Liquid Limit and linear shrinkage)	Liquid limit, linear shrinkage, estimated Plasticity Index and Plastic limit	Compacted density, optimum moisture content, California Bearing Ratio.
Rock quality	Coarse particle strength	Behaviour during construction
Water quality (Salt content in ppm)	Suitability for use in construction	Risk of environmental concerns

6. PARAMETERS AND PREDICTIONS

The software will also calculate a number of parameters for use in assessing the suitability of a material(s) or mix. Various authorities have set limits to one or more of the parameters and a user should seek out those appropriate for a particular location.

The software will then use these parameters to predict other properties, maximum modified compacted dry density (MDD), optimum moisture content for compaction (OMC) and California Bearing Ratio (CBR).

The data used to develop predictions was collected in Australia (Ingles & Metcalf: 1972), The maximum dry density and the optimum moisture content were the dependent variables. All defined variables were used as independent variables in a stepwise forward variable selection procedure for the linear regression analysis to determine best models for predicting the maximum dry density, MDD, and optimum moisture content, OMC. Microsoft Excel and SPSS software were used for the statistical analysis.

The initial results of the linear regression analysis are indicated below, Figure 1.

Maximum Modified Dry Density

$$\text{MDD (t/m}^3\text{)} = 2.0513 - 0.0114*\text{PL} - 0.0002*\text{PM} + 0.2901*\text{GR}_2$$

Where; PL = Plastic Limit, PM = Plasticity Modulus = Ip x % pass 0.425, GR_2 = % pass 0.075 / % pass 0.425 and,

R squared = 0.81, Standard Error = 0.074(t/m^3), Observations 25

It should be noted that the equation exhibits moderate to strong fit with the given data and provides only an estimate of MDD.

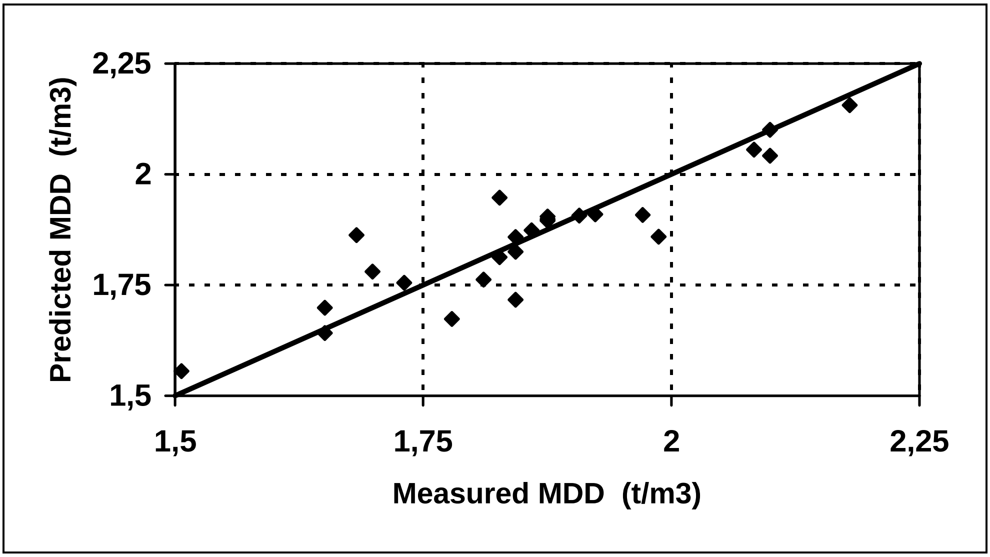

Figure 1: Predicted MDD vs. Measured MDD

Material classification

The Unified Soil Classification has been adopted for this Test Kit because it provides broad estimates of compacted density and CBR for the various classes. These classes (e.g. GW – well graded gravel) are assigned in the spreadsheet to again provide guidance in assessing any new material source. The estimates of properties should be confirmed by further appropriate laboratory testing before being used directly in construction.

7. CONCLUSIONS

Low volume rural roads play a vital role in the economic viability of the rural sector. They are predominately unsealed two-lane two-way roads.

Increasing use of readily available local materials, often of marginal quality, is required for cost-effective construction and maintenance of low volume roads.

The purpose of this project is to develop a simple and inexpensive means of conducting at least an initial evaluation of new source materials. The specific objective is to produce a man portable road base test kit suitable for use by non-professional employees.

The process has been to assemble a simple testing equipment package, now being trialed with local government technical staff using local materials.

The Road Base Test Kit together with the procedures, calculations and estimates are not equivalent to a full scale professional testing laboratory evaluation. However, in the absence of such an evaluation the Road Base Test Kit will provide a first estimate on which to make decisions as to the use of available materials.

These estimates must be used with caution as they are based on statistical analyses still under development which may or may not be entirely valid for a specific set of conditions.

The results are input directly into a spreadsheet calculation to calculate the required material parameters, grain size distribution and plasticity, the USC classification and the ‘fit’ to both the Wooltorton and Paige-Green plots. The software will also estimate the maximum modified compacted dry density and moisture content using equations under development by ARRB.

A specific and important feature is that using the ARRB web site to calculate the parameters will also automatically archive the data at ARRB for future use to refine the predictive equations.

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Plate 1 Road Base Test Kit

The Road Base Test Kit equipment

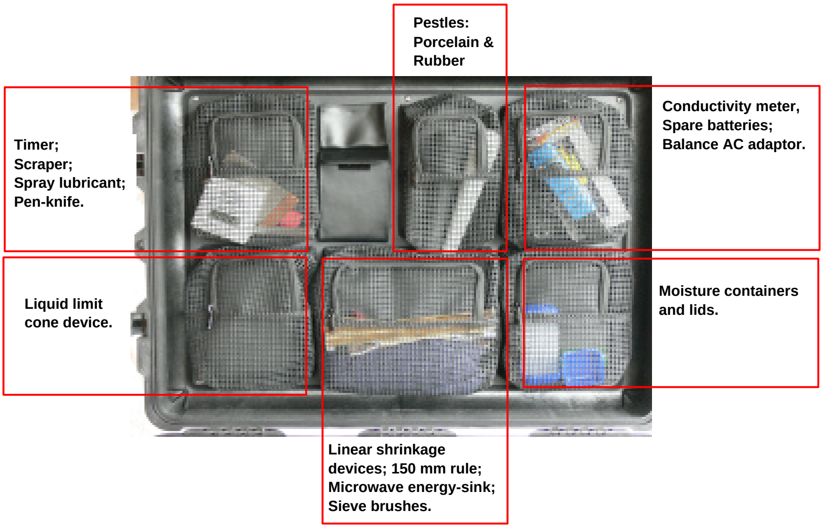
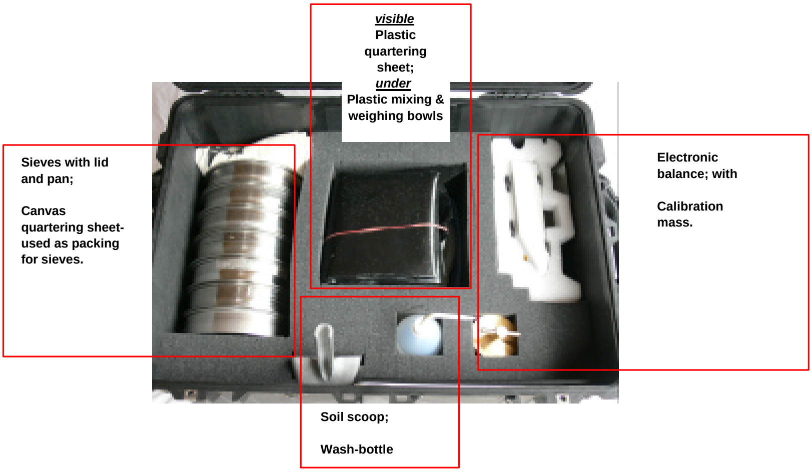


Plate 2 Road Base Test Kit

The Road Base Test Kit equipment



Appendix 1 Example of spreadsheets under development

Road Base test kit - data input sheet

Sample number		sampled	by		date	
GPS Location		rainfall				
source						
description						
		tested	by		date	

Particle size distribution

		Input data	
		grams	
Mass of sample before sieving	m1	1000.0	
sieve		**Mass retained**	
37.5 mm	m2	0.0	
19 mm	m3	0.0	
9.5 mm	m4	30.0	
4.75 mm	m5	660.0	
2.36 mm	m6	110.0	
0.425 mm	m7	30.0	
0.075 mm	m8	40.0	
mass passing 0.075 mm	m9	60.0	
check sum		930.0	RETEST
Liquid limit	mwl	36.8	
LL	mdl	30.0	
Penetration	mm	20.0	
Linear Shrinkage			
LS	mm	1.6	
Plastic limit	mwp	35.4	
PL	mdp	30.0	
Water quality	ppm	3200	
Stone quality			

Road Base test kit - material parameter sheet

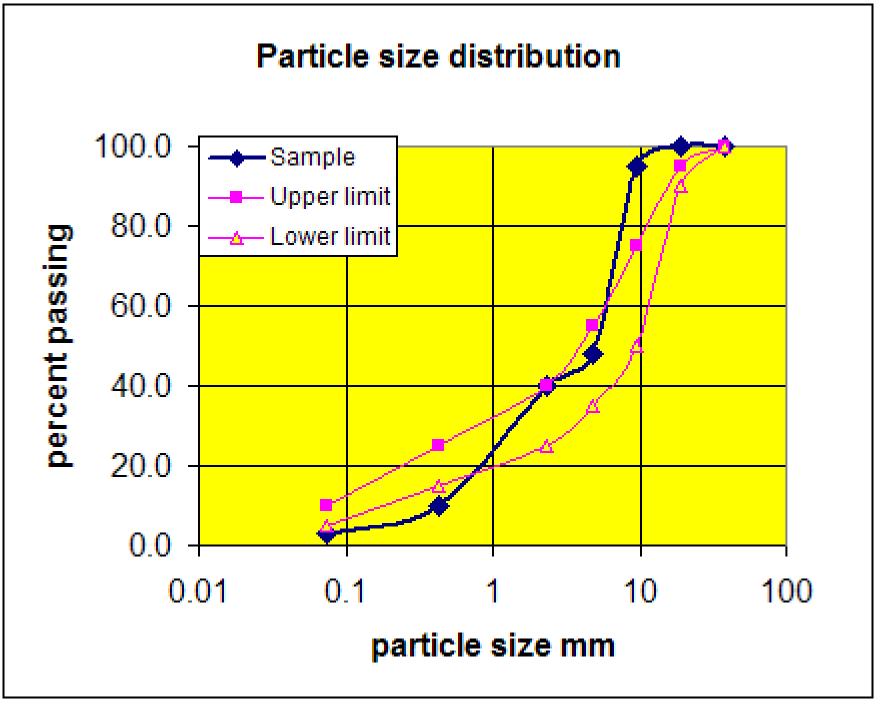
Sample number [ ] sampled by [ ] date [ ]

GPS Location [ ] rainfall [ ]

source [ ]

description [ ]

tested by [ ] date [ ]



Parameters			Specifications	
Particle size distribution			**Target criteria**	
size mm	**% passing**		**size mm**	**% passing**
37.5	100.0		**100**	**100**
19	100.0		95	90
9.5	95.0		75	50
4.75	48.0		55	35
2.36	40.0		40	25
0.425	10.0		25	15
0.075	3.0		10	5
Liquid limit	20.0	**LL**	14 to 20	
Linear Shrinkage	6.0	**LS**	6 to 12	
Plasticity index	12.8	**Ip**	4 to 8	
Plastic Limit	6.7	**PL**	6 to 16	
Subgrade CBR	not valid		50 to 60	
Water quality	use with care			
Stone quality				

Road Base test kit - mix design sheet

Percentage Passing Sieve Size		your specification max	min					
Sieve Size (mm)	Log size (mm)	limit %	limit %	A	B	C	Target	Design
37.5	1.57403	100	100	100.0	100	100	100	100.0
19	1.27989	95	90	100.0	90	95	92.5	95.0
9.5	0.97886	75	50	95.0	65	95	62.5	84.8
4.75	0.67669	55	35	48.0	25	90	45	54.0
2.36	0.37291	40	25	40.0	10	85	32.5	44.7
0.425	-0.37161	25	15	10.0	2	80	20	30.4
0.075	-1.12494	10	5	3.0	1	50	7.5	17.8
			Ip	12.8	0	20		10.8174

Mix

A	B	C
33.0	34.0	33.0

Total

Inputs

Maximum and minimum limits to the design % passing

% passing sieves for A - automatic input from 'Material testing'.

% passing for B - manual input; % passing for c - manual input

% passing for actual mix ABC

% passing design mean

Solve

Outputs

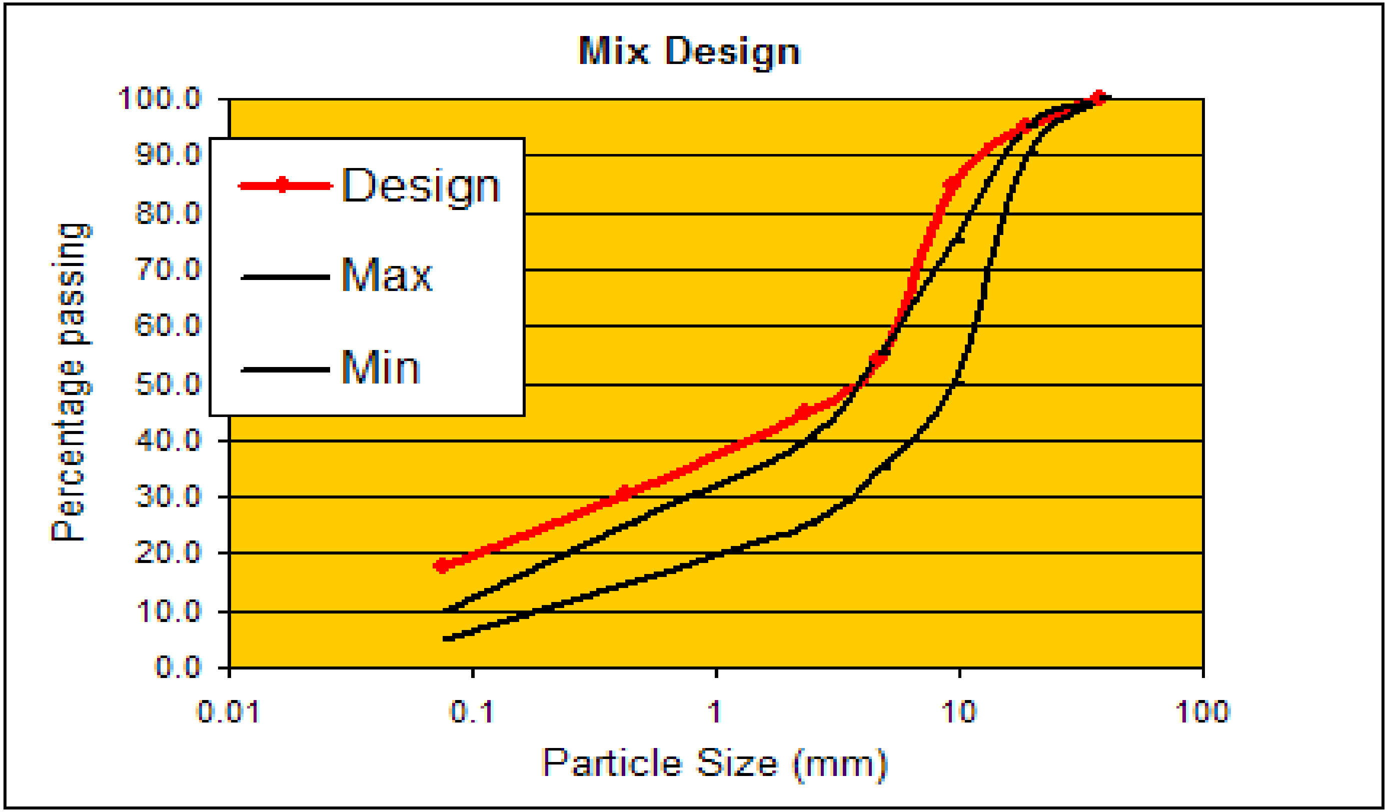
Mix proportions

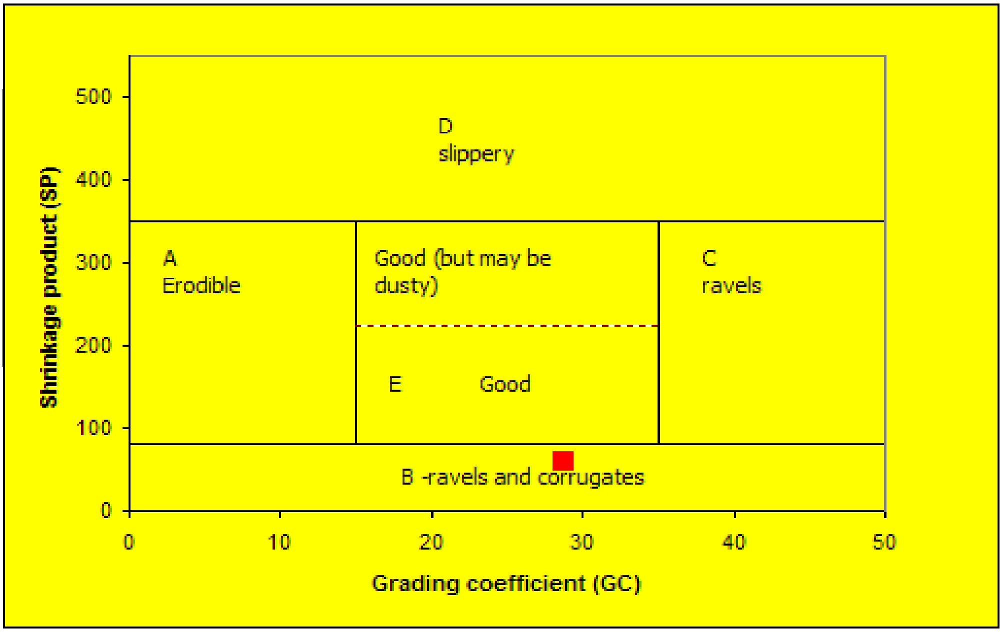
Actual % passing each sieve for mix

Plasticity index

Mix Ip **4**

50


Mix Design
Percentage passing
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Design
Max
Min
0.01
0.1
1
10
100
Particle Size (mm)



Paige-Green plot

	GC	SP
Data	**28.80**	**60.00**

Output	
Erodible	
Ravels and corrugates	Yes
Ravels	
Slippery	
Good	

Unified Soil Classification

size mm	% passing
37.5	100.0
19	100.0
9.5	97.0
4.75	31.0
2.36	20.0
0.425	17.0
0.075	13.0

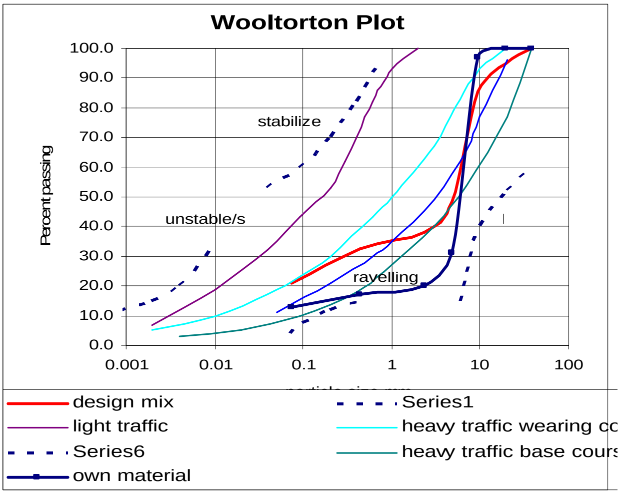
Liquid limit LL		21.0
Linear Shrinkage LS		1.6
		sw
Plasticity index Ip		3.4
Plastic Limit PL		17.9

USC classification

0

0

Silty gravel, GM, possible density 120-145 pcf, possible CBR 20-80



“Highway and Bridge Engineering 2008”, International Symposium
Iasi, România, December 12, 2008

Road Rehabilitation by Using the Multi-Criterial Analysis

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Summary

This paper presents the rehabilitation possibilities of a 20.0 km long district road, with access in the area of Cavnic – Ocna Sugatag, by the method of choosing the optimum intervention solution for an existent road, based on relevant aspects regarding technical, economical and environmental parameters.

The surveyed factors are: the workings` cost for each planned solution, the impact on the environment and the tourist attraction in the region.

There are presented two possible solutions:

- only repairs, in order to ensure the traffic;

- rehabilitation of the road, in terms of placing it in geometrical items and the traffic’s requirements, in a 10 years perspective.

KEYWORDS: road, analyse, costs, design solutions, environmental impact, tourism.

1. INTRODUCTION

In order to improve the traffic’s standards in Romania, the trend is to create high-speed roads (highways and express roads), to link different attraction areas in Romania, or to link Romania to neighbor countries.

However, the final destinations of the traffic’s participants are points which are set near or far to these high-speed roads, depending on the economical or touristy interest. These “side-roads” are existing roads (national, district and communal); therefore their problems have to be solved, as well as the traffic conditions.

The development of the public roads` net in the N-W area of Romania is showed in “The Development of the Regional Strategically Frame in the N-W of Romania for the years 2007 to 2013”. According to this document and the HG 28/2008, in order to work out the double study of a paper, it has to be considered the specific environmental implications, the rehabilitation’s cost of the designed solution, beside the technical aspects of the road itself and its efficiency regarding the interest for the tourist attractions of the region.

These criteria are unlimited; there are also other elements which could contribute to the regional development and tourist attraction.

The theoretical presentation and the case analysis will consider following multi-criterial analysis items:

- the cost

- the environmental impact

- the tourist attraction in the region

Each handled criterion will be awarded with 0 to 100 points, and will be appreciated as having a certain amount in promoting the work solution.

Each criterion will get a fair share, but it could weight differently according to the investor's interests (Table 1).

Table 1. Criterion

Criterion	**Percentage (%)**
Cost	33.34
Environmental impact	33.33
Tourist attraction in the region	33.33
Total	**100**

For the case study there have been considered the rehabilitation possibilities of a 20.0 km long district road with access in the area of Cavnic – Ocna Sugatag.

Alternatives to be analysed are following two options:

- Option I "no project", meaning only the existent road's reparations for traffic insurance without interventions on water leakage, or the replacement of footbridges, or bringing the bridges to E Class loading;

- Option II "the existent road's rehabilitation", meaning the road system's sizing of the future traffic, the lay-out of the ditches according to the road's gradient, the water evacuation's insurance, the earthwork's strengthening and bringing the bridges to E Class loading.

The score for each variant was counted and the best one, from the criteria ensemble's point of view, was chosen.

2. COST

This criterion was chosen because it reflects the best the investitional effort, by giving at the same time the costs` amount for each analysed alternative.

In order to evaluate the two possible variants, the main work volume which is needed to be done was counted:

- earthworks
- footbridges
- road system
- bridges
- earthwork strengthening works
- hydro technical works

The score for every variant will be established by following formula:

$$\frac{Cost_{min}}{Cost_{option\ i}} \times 100 \qquad (1)$$

3. THE ENVIRONMENTAL IMPACT

Following criteria have been considered for the comparative analysis of the studied options:

1. Quality of the air
2. Noise level
3. Protected areas
4. Surface water

3.1. Quality of the air

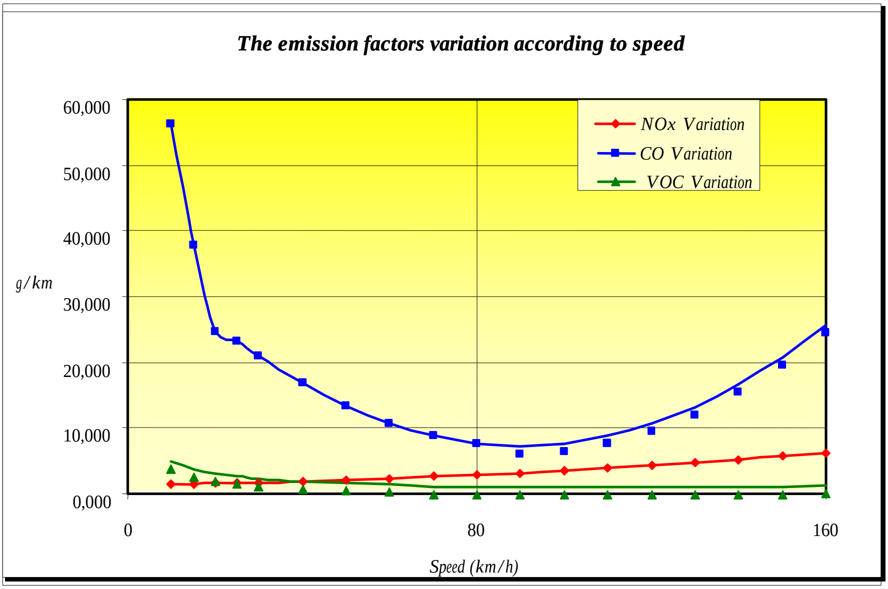

Figure 1. The variation of the emission factors according to speed

The quality of the air will be influenced by the gas emissions from the vehicles during driving. Thus, the continuous driving flow (without repeated braking and accelerations), is decreasing the gas emissions in the atmosphere.

Generally, the emission of the substances that pollute the air according to vehicles` drives speed increases in traffic jams (see Figure 1).

Thus, studies show that:

- the CO emissions increase 1.5-2.0 times during the acceleration-breaking cycles, and up to 25 times while low speed driving

- the emission concentration of hydrocarbons is minimal at constant speed, it slowly increases by acceleration and increases up to 20 times in low speed driving

- the emission concentration of NO_x increases while speeding

The polluting substances which are let out in the atmosphere are transferred into other environmental factors: water, soil, vegetation.

3.2. Noise Level

By the execution of the new covering, the repeated accelerations and breakings with straight impact on the noise level will be eliminated. The noise made by light vehicles is less influenced by the road’s condition. However, the acoustic power let out by heavy vehicles is highly influenced by the state of the surface’s road.

Another analyzed aspect was the crossing of inhabited areas.

Generally, according to the present law, the maximum permitted level (50 dB) is reached.

The covering’s modernization, along with the few corrections of the line, will lead to this level’s decrease.

3.3. Protected areas

Protected areas are the natural parks (protected flora and fauna species). Other protected areas are those situated in historical places.

The most harmed areas could be the forests, in our case both flora and fauna being rich and diversified.

3.4. Surface waters

Surface waters, due to precipitations, will be collected in designed ditches and will be led to an emissary. Then they will be poured by a decantation system.

These measures have been taken in order to avoid the present situation when waters stop flowing in the road area. This phenomenon favoring the circumstance of cramming in the freeze-defreeze times.

In Option II the flooding areas will be emptied by designed depth ditches drains.

4. THE TOURIST ATTRACTION IN THE REGION

This criterion promotes the increasing potential of tourist destination interests in the region Cavnic – Ocna Sugatag, by getting an easy access to the tourist attractions of the area.

The construction of access roads to the tourist attractions (churches, ski slopes, curative zones) will lead to increased tourism in the region, and also to the necessity of building more hostel places.

This will have a positive impact upon maintaining the young active population in the area, leading towards to the economic development of the region.

By this criterion it will be evaluate the road's efficiency from the tourist interest point of view in both options.

The score will be established by calculating the ratio between the minimum time needed for reaching every tourist objective (Tp_{min}) on the road, and the current time needed to arrive at the destination ($Tp_{current}$).

Thus, the score will be established by following the next formula:

$$\frac{Tp_{\min}}{Tp_{current}} \times 100 \qquad (2)$$

5. SCORE

5.1. The Cost

Option I 16.28 mil lei

Option II 38.27 mil lei (Table 2)

Table 2. Cost

Criterion	Value		Percentage %	
	Option I	**Option II**	**Option I**	**Option II**
Cost	100%	43%	33.34	14.34

5.2. The Environmental Impact Quantification

In order to compare the studied options, the environmental impact will be measured by using the above presented criteria.

The quantification of the environmental impact was made by using a notation scale, from "-3" to "+3", for each criterion as follows:

- "-3" - important negative impact which requires redesigning or giving up the project

- "-2" - important negative impact which can be minimized by taking adequate measures

- "-1" - less important negative impact which can be minimized by taking adequate measures

- "0" - no impact whatsoever

- "+1" - reduced positive impact

- "+2" - important positive impact

- "+3" - very important positive impact

In the end, the marks for all criteria will be counted and the option's mark will be obtained. The score for each option will be calculated as follows:

- 0 for the marks summing "-10" or less

- (the sum of the marks +10) x5 for the marks summing more than "-10" and less than 10

- 100 for the marks summing more or equal to 10 (Table 3).

Table 3. Environmental impact quantification

Criterion	Option I	Option II
1. quality of the air	-3	+3
2. noise level	-3	+3
3. protected areas	-3	+3
4. surface water	0	0
TOTAL	**-9**	**+9**

Thus, the points for the two solutions are: 5 points for Option I and 95 points for Option II.

5.3. The Tourist Attraction in the Region

- 20.0 km : 25km/h = 0.80h in Option I

- 20.0 km : 60km/h = 0.34h in Option II (Table 4)

Table 4. Tourist attraction in the region

Criterion	**Sector Length**		**Percentage %**	
	Option I	**Option II**	**Option I**	**Option II**
Tourist attraction in the region	42.50%	100%	14.16	33.33

5.4. Final Score

Table 5. Final score

	Partial Score		**Percentage (%)**	**Final Score**	
	Option I	**Option II**		**Option I**	**Option II**
Cost	100	43	33.34	33.34	14.34
Environmental impact	5	95	33.33	1.67	31.66
Tourist attraction in the region	42.5	100	33.33	14.16	33.33
Total			**100**	**49.17**	**79.33**

6. CONCLUSIONS

The utilization of the multi-criterial analyse, used in the elaboration doable studies, invests the chosen solution an increased technical-economical efficiency and a proportion in situating it in social, economical and environmental development in the located area.

In this case the recommended solution is Option II – “the rehabilitation of the existent road”.

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The Consolidation of Steel Bridges Superstructures by pre-stressing

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Summary

As the working load increase causes overloading, it is mostly inefficient to consolidate the steel decks of bridges by increasing the girder section attaching new elements. (using great amount of steel, the increase of the bearing capacities is low; see the work: The Consolidation of Steel Bridges Superstructures Without Introducing Initial Stress States).

The more the allowable stress of steel is consumed by permanent load, the more this consolidation is inefficient..

Better solutions of consolidation are obtained when an initial stress state is introduced to act contrary to the stress state produced by loads.

The following consolidation solutions have been taken into consideration:

1. The consolidation by enhancing the base of the girder section with chord plates applied directly on the base of the pre-flexion girder

2. The consolidation with pre-stressed steel rod

KEYWORDS: steel bridge floor, consolidation, consolidation chord plates, pre-flexion, pre-stressed steel rod.

Notation:

- $y_n^s ; y_n^i$: the distance from the section centroid of the unconsolidated girder section to the top fibre/bottom fibre;
- $y_c^s ;\ y_c^i$: the distance from the section centroid of the consolidated girder section to the top fibre/bottom fibre;
- G_n : the section centroid of the unconsolidated girder section;
- G_c : the section centroid of the consolidated girder section;
- Δt : the thickness of the consolidation chord plates applied on the base of the girder section;
- l_t : the length of the consolidation tension rod;

- e : the distance from the section centroid of the consolidation steel tension rod to the inferior base of the girder;
- I_{gn} ; I^{b}_{gn} : the moment of inertia (second moment of area) of the unconsolidated net/rough girder section;
- I_{gc} : the moment of inertia (second moment of area) of the consolidated net girder section;
- A_{gn} ; A^{b}_{gn} : the area of the unconsolidated net/rough girder section;
- A_{t} : the area of the consolidation pre-tension rod;
- R : the girder pre-flexion force;
- X_{1} : the self-tension axial stress from the consolidation tension rod;
- X_{2} : the pre-tension axial stress from the consolidation tension rod;
- M_{gn} : the maximum bending moment given by the weight of the unconsolidated structure;
- $M_{g'}$: the maximum bending moment given by the weight of the consolidation elements;
- M_{p} : the bending moment given by the pre-flexion;
- M_{u} : the maximum bending moment given by traffic loads;
- M_{m} : the weighted average value of the bending moment on the tension rod consolidation length, given by traffic and permanent loads;
- M_{x_1} : the bending moment in the girder given by X_1;
- M_{x_2} : the bending moment in the girder given by X_2;
- s^{s}_{gn} ; s^{i}_{gn} : the normal unit stress produced by M_{gn} on the unconsolidated girder section at the top fibre/bottom fibre;
- $s^{s}_{g'}$; $s^{i}_{g'}$: the normal unit stress produced by $M_{g'}$ on the unconsolidated girder section at the top fibre/bottom fibre;
- s^{s}_{pn} ; s^{i}_{pn} : the normal unit stress produced by M_p on the unconsolidated girder section at the top fibre/bottom fibre;
- s^{s}_{pc} ; s^{i}_{pc1} ; s^{i}_{pc2} : the normal unit stress produced by M_p on the consolidated girder section at the top fibre/bottom fibre(in the points 1 and 2);
- s^{s}_{uc} ; s^{i}_{uc1} ; s^{i}_{uc2} : the normal unit stress produced by M_u on the consolidated girder section at the top fibre/bottom fibre(in the points 1 and 2);
- s^{s}_{c} ; s^{i}_{c} ;(s^{i}_{c1} ; s^{i}_{c2}): the total normal unit stress produced on the consolidated girder section at the top fibre/bottom fibre(in the points 1 and 2);

- $s_{I}^{s}; s_{I}^{i}$: the normal unit stress produced by X_2 on the consolidated girder section at the top fibre/bottom fibre;
- $s_{II}^{s}; s_{II}^{i}$: the normal unit stress produced by the exploitation load and X_1 at the top fibre/bottom fibre;
- s_{an}: allowable normal stress of the steel from the unconsolidated girder;
- s_{ac}: allowable normal stress of the steel from the consolidation elements;
- s_{a}^{t}: allowable normal stress of the steel from the consolidation pre-tension rod;

1. INTRODUCTION

Below are presented two consolidation solutions obtained by pre-stressing, for the main simple web girders of a bridge superstructure with an exceeding bearing capacity and a case study in which the methods used are being explained.

2. CONSOLIDATION METHODS

2.1. The consolidation by enhancing the base of the girder section with chord plates applied directly on the base of pre-flexion girder

From a technological point of view the working stages are as follows:

- scaffoldings are placed under the girders which are stressed (the pre-flexion) using presses applied on the scaffoldings;
- the rivet heads from the inferior base of the girder are cut (without taking out the cut rivets) on the area on which new steel plates are to be attached;
- the new steel plates are placed in the correct position, regarding the position of the existing rivets;
- the cut rivets are taken out one by one and are replaced with the new ones, which are installed in the rectified holes;

As regards the calculation, it results the following the following stress states result, which when combined give the final girder stress state:

a) On girders' unconsolidated section develops a stress state produced by the bending moment given by the weight of the unconsolidated structure and of the new introduced elements and the bending moment given by pre-flexion (the initial stress state) (Figure 1);

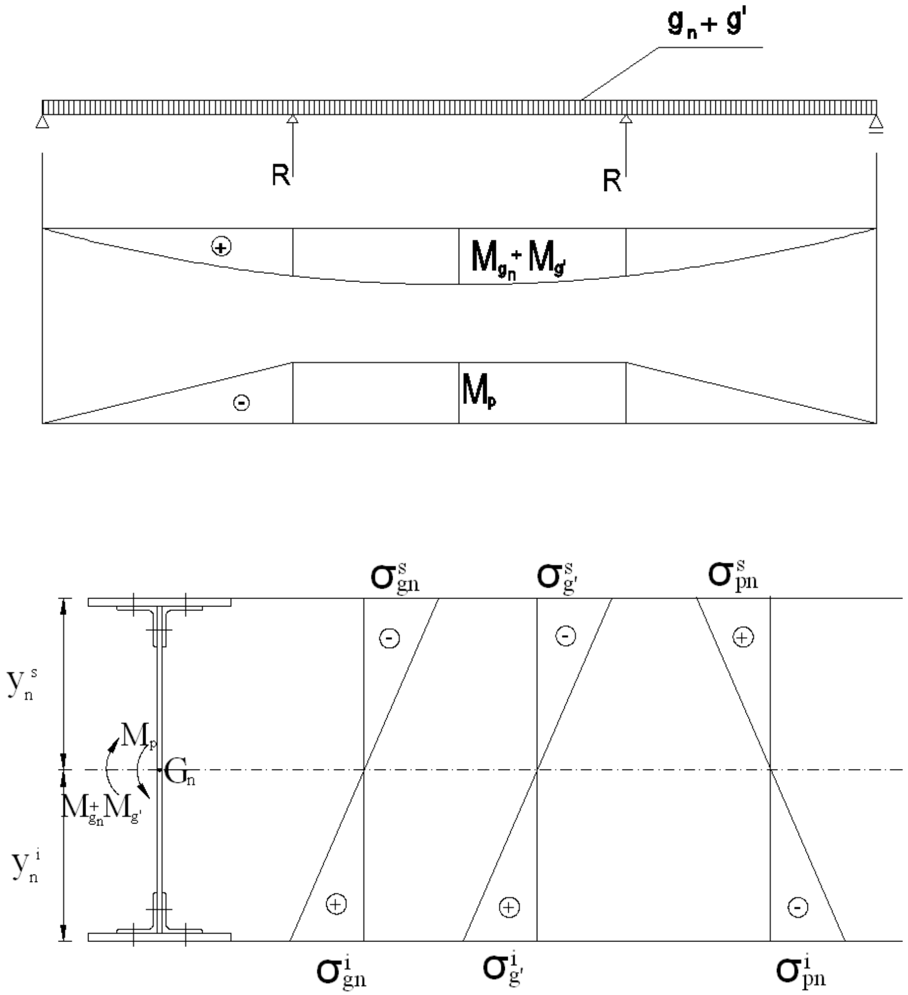
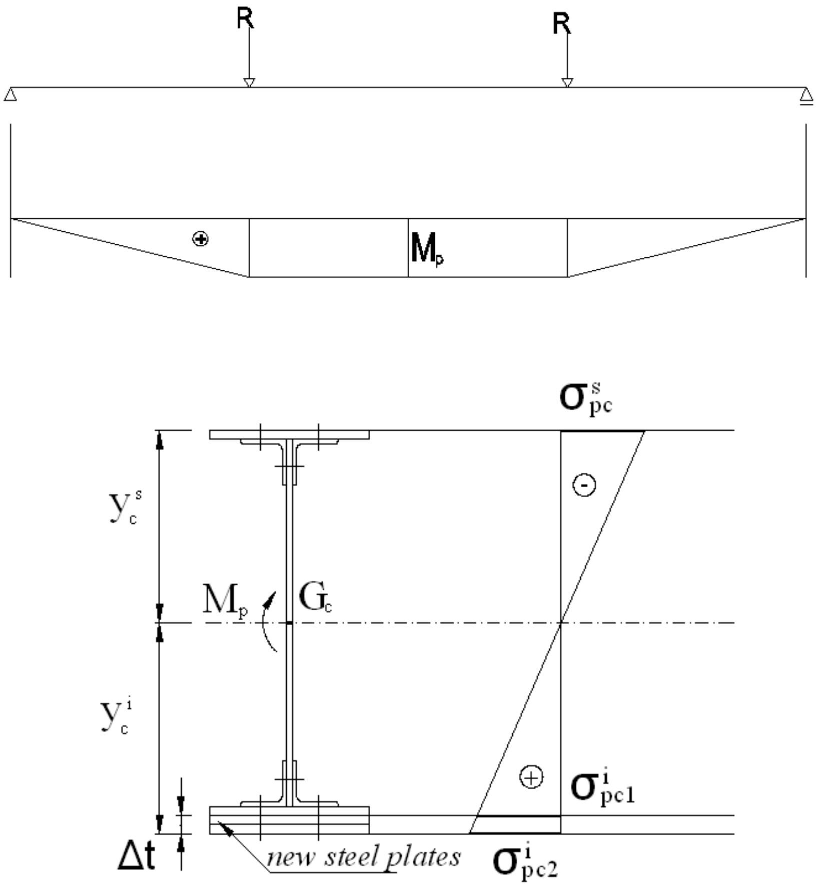


Figure 1.

$$
\begin{aligned}
s^{s}_{gn} + s^{s}_{g'} - s^{s}_{pn} &= \frac{M_{gn} + M_{g'} - M_{p}}{I_{gn}} y^{s}_{n} \\
s^{i}_{gn} + s^{i}_{g'} - s^{i}_{pn} &= \frac{M_{gn} + M_{g'} - M_{p}}{I_{gn}} y^{i}_{n}
\end{aligned}
\qquad (1)
$$

b) After the chord plates have fastened on the pre-stressed structure, the presses are removed, which is equivalent to load the girders with the pre-flexion forces R;

the consolidated girder section takes over the bending moment given by R forces. (Figure 2);



$$s^{s}_{pc} = \frac{M_p}{I_{gc}} y^s_c$$

$$s^{i}_{pc1} = \frac{M_p}{I_{gc}} (y^i_c - \Delta t) \qquad (2)$$

$$s^{i}_{pc2} = \frac{M_p}{I_{gc}} y^i_c$$

c) The consolidated structure is put in use - the consolidated girder section takes over the bending moment given by traffic loads. (Figure 3);

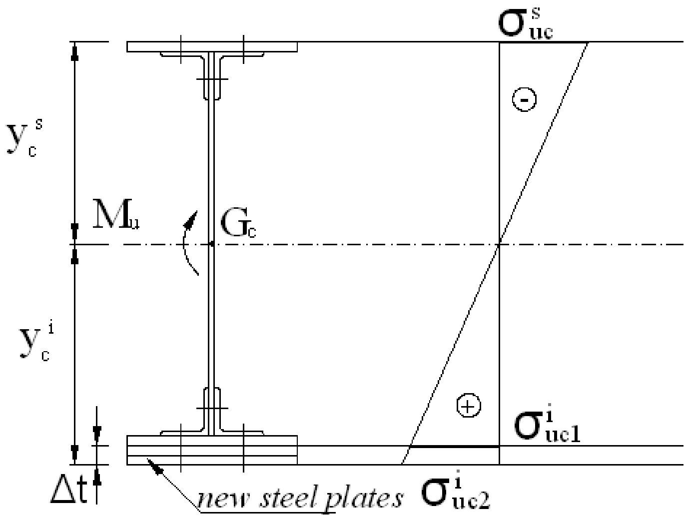


Figure 3.

$$s^{s}_{uc} = \frac{M_u}{I_{gc}} y^s_c$$

$$s^{i}_{uc1} = \frac{M_u}{I_{gc}} (y^i_c - \Delta t) \tag{3}$$

$$s^{i}_{uc2} = \frac{M_u}{I_{gc}} y^i_c$$

d) The final stress state on the consolidated section (Figure 4);

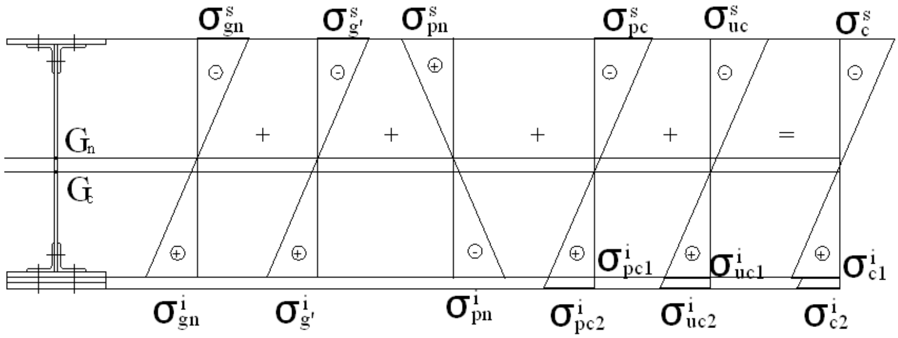


Figure 4.

The strength condition for the consolidated structure is:

$$
\begin{aligned}
&s_{c}^{s} = s_{gn}^{s} + s_{g'}^{s} - s_{pn}^{s} + s_{pc}^{s} + s_{uc}^{s} \leq s_{an} \\
&s_{c1}^{i} = s_{gn}^{i} + s_{g'}^{i} - s_{pn}^{i} + s_{pc1}^{i} + s_{uc1}^{i} \leq s_{an} \\
&s_{c2}^{i} = s_{pc2}^{i} + s_{uc2}^{i} \leq s_{ac}
\end{aligned}
\tag{4}
$$

2.2. The consolidation with pre-stressed steel rod

One or several steel pre-tension rods which will introduce an advantageous initial stress state for the structure, will be attached to the unconsolidated main girders.

The simplest solution suggests to introduce a rectilinear steel tension rod is introduced under the inferior base of the main girders.

Inside the steel tension rod which is being applied develops a tensile force, produced by the traffic loads (self-tensile force), which is determined on the girder-tension rod structure once statically indeterminate. (Figure 5);

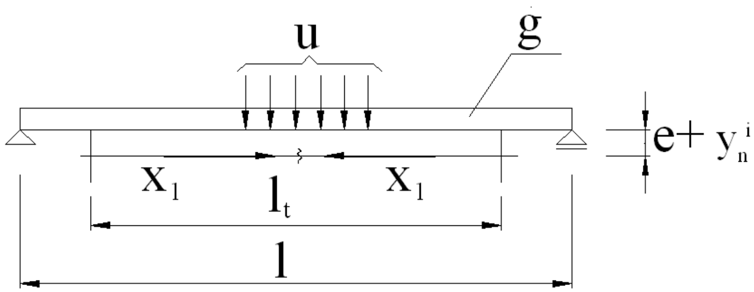


Figure 5.

The expresion for X_1 is:

$$X_1 = \frac{M_m}{\left(e + y_n^i\right) + \frac{I_{gn}^b}{\left(e + y_n^i\right)}\left(\frac{1}{A_{gn}^b} + \frac{1}{A_t}\right)} \quad (5)$$

The relation (5) will be used for any position of the permanent and traffic loads on the structure if a a weighted average value M_{um} is considered for the bending moment they produce on the girder tension rod consolidation length, in these conditions the free term ? $_{1p}$ of the static balance equation:

$$X_1 d_{11} + \Delta_{1p} = 0$$

has invariable form.

Knowing X_1, X_2 will be determined:

$$X_2 = A_t s_a^t - X_1 \quad (6)$$

The stress states from the structure are the following:

a) The stress state from the girder in the rod pre-tension stage (stage I), produced by X_2 (Figure 6);

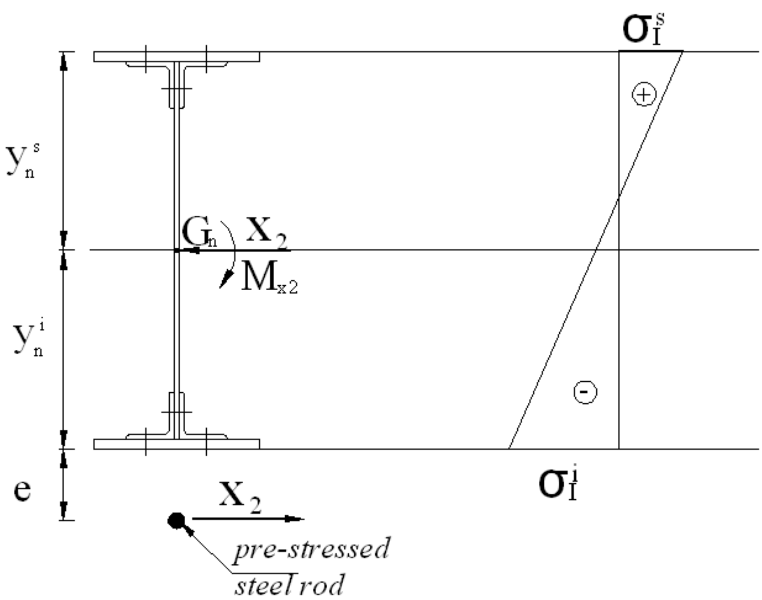


Figure 6.

$$M_{x2} = X_2\left(e + y_n^i\right)$$
$$s_{\mathrm{I}}^{s} = -\frac{X_2}{A_{gn}} + \frac{M_{x2}}{I_{gn}} y_n^s \qquad (7)$$
$$s_{\mathrm{I}}^{i} = -\frac{X_2}{A_{gn}} + \frac{M_{x2}}{I_{gn}} y_n^i$$

b) The stress state from the pre-stressed girder, given by permanent and traffic loads and X_1 (stage II);

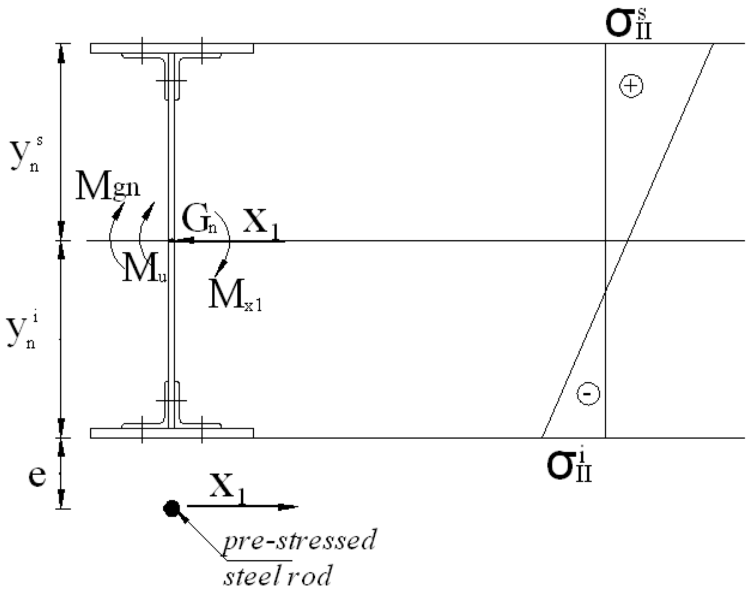


Figure 7.

$$M_{x1} = X_1\left(e + y_n^i\right)$$
$$s_{\mathrm{II}}^{s} = -\frac{M_{gn} + M_u}{I_{gn}} y_n^s - \frac{X_1}{A_{gn}} + \frac{M_{x1}}{I_{gn}} y_n^s \qquad (8)$$
$$s_{\mathrm{II}}^{i} = -\frac{M_{gn} + M_u}{I_{gn}} y_n^i - \frac{X_1}{A_{gn}} + \frac{M_{x2}}{I_{gn}} y_n^i$$

c) The final stress state in the consolidated girder results:

$$s_c^s = s_{\mathrm{I}}^s + s_{\mathrm{II}}^s \le s_{an}$$
$$s_c^i = s_{\mathrm{I}}^i + s_{\mathrm{II}}^i \le s_{an} \qquad (9)$$

The stress state in the structure modifies if the tension loss caused by braces slidings and rod steel relaxation are taken into consideration.

3. CASE STUDY

The two consolidation methods are applied for the main girder of a bridge with the following characteristics (Figure 8):

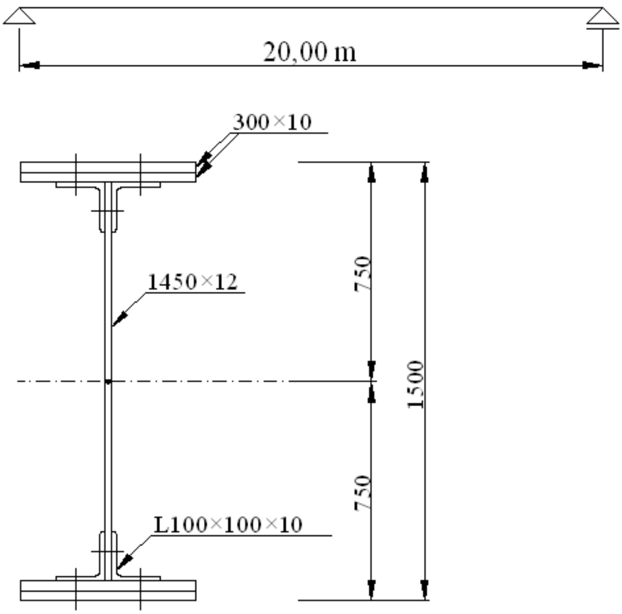


Figure 8.

$$M_{gn} = 1112 KNm$$

$$M_u = 1440 KNm$$

$$s_{an} = 150 \,{}^{N}\!/\!{}_{mm^2}$$

$$A_{gn}^{b} = 37080 mm^2$$

$$A_{gn} = 34320 mm^2$$

$$I_{gn}^{b} = 1340240 \times 10^4 mm^4$$

$$I_{gn} = 1191138 \times 10^4 mm^4$$

The maximum unit stress of the girder produced by the bending moment given by the permanent and traffic loads is up to 160,8 N/mm^2.

3.1. The consolidation according to 2.1.

The inferior base of the girder is consolidated with two 300×10 mm steel plates (Figure 9) from OL 37.2 (s_{ac}=145 N/mm^2) steel.

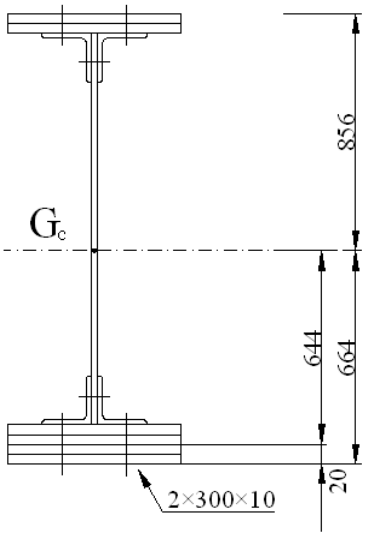


Figure 9.

$$M_{g'} = 23{,}55 KNm$$

$$I_{gc} = 1446893 \times 10^4 mm^4$$

The pre-flexion is performed by two forces R=200 KN symmetrically aplied on the structure, at 5 metres from the bearings.

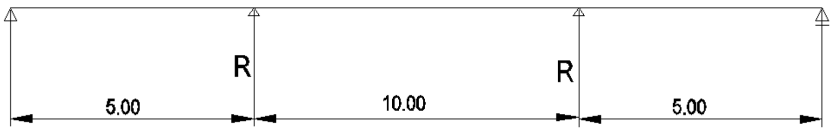


Figure 10.

$$M_p = 1000 KNm$$

The following values are obtained with (1), (2), (3) and (4):

$$s_{gn}^{s} = s_{gn}^{i} = \frac{1112 \times 10^6}{1191138 \times 10^4} 750 = 70{,}1 \, {}^{N}\!/_{mm^2}$$

$$s_{g'}^{s} = s_{g'}^{i} = \frac{23{,}55 \times 10^6}{1191138 \times 10^4} 750 = 1{,}5 \, {}^{N}\!/_{mm^2}$$

$$s_{pn}^{s} = s_{pn}^{i} = \frac{1000 \times 10^6}{1191138 \times 10^4} 750 = 63{,}0 \, {}^{N}\!/_{mm^2}$$

$$s_{pc}^{s} = \frac{1000 \times 10^6}{1446893 \times 10^4} 856 = 59{,}2 \, {}^{N}\!/_{mm^2}$$

$$s_{pc1}^{i} = \frac{1000 \times 10^6}{1446893 \times 10^4} 644 = 44{,}6 \, {}^{N}\!/_{mm^2}$$

$$s_{pc2}^{i} = \frac{1000 \times 10^6}{1446893 \times 10^4} 664 = 45{,}9 \, {}^{N}\!/_{mm^2}$$

$$s_{uc}^{s} = \frac{1440 \times 10^6}{1446893 \times 10^4} 856 = 85{,}2 \, {}^{N}\!/_{mm^2}$$

$$s_{uc1}^{i} = \frac{1440 \times 10^6}{1446893 \times 10^4} 644 = 64{,}1 \, {}^{N}\!/_{mm^2}$$

$$s_{uc2}^{i} = \frac{1440 \times 10^6}{1446893 \times 10^4} 664 = 66{,}1 \, {}^{N}\!/_{mm^2}$$

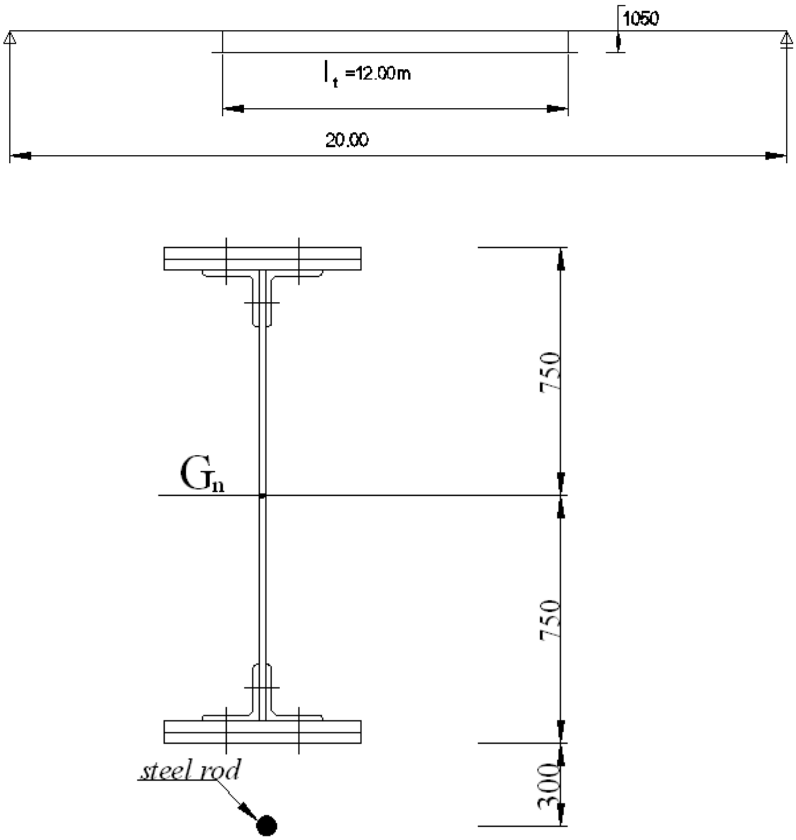
$$s_{c}^{s} = 70{,}1 + 1{,}5 + 63{,}0 + 59{,}2 + 85{,}2 = 153{,}0 \, {}^{N}\!/_{mm^2} < s_{an} + 3\% = 154{,}5 \, {}^{N}\!/_{mm^2}$$

$$s_{c1}^{i} = 70{,}1 + 1{,}5 - 63{,}0 + 44{,}6 + 64{,}1 = 117{,}3 \, {}^{N}\!/_{mm^2} < s_{an} = 150{,}0 \, {}^{N}\!/_{mm^2}$$

$$s_{c2}^{i} = 45{,}9 + 66{,}1 = 112{,}0 \, {}^{N}\!/_{mm^2} < s_{ac} = 145{,}0 \, {}^{N}\!/_{mm^2}$$

3.2. The consolidation according to 2.2.

The inferior base of the girder is consolidated with pre-stressed steel rod consisting of 24 Ø7mm SBP I wires, located under the girder base at a distance of e = 300 mm (Figure 11).



$$A_t = 24\frac{\pi \times 7^2}{4} = 923,6mm^2$$

$$\sigma_{at} = 1000 \, N/mm^2$$

$$M_m = 2024KNm$$

The length on which the girder is consolidated (the length of the consolidation tension rod) is l_t = 12.00 m.

X_1 is calculated with relation (5):

$$X_1 = \frac{2024\times10^6}{(300+750)+\frac{1340240\times10^4}{300+750}\left(\frac{1}{37080}+\frac{1}{923,6}\right)} = 133030N$$

X_2 is calculated with relation (6):

$$X_2 = 923,6\times1000-133030 = 790570N$$

The normal unit stress produced by X_2 in pre-tension stage is determined with relations (7):

$$s_{\mathrm{I}}^{s} = -\frac{790570}{34320}+\frac{790570(300+750)}{1191138\times10^4}750 = 29,3\, {N}/{mm^2}$$

$$s_{\mathrm{I}}^{i} = -\frac{790570}{34320}-\frac{790570(300+750)}{1191138\times10^4}750 = -75,4\, {N}/{mm^2}$$

The normal unit stress produced by exploitation load and X_1 is determined with relations (8):

$$s_{\mathrm{II}}^{s} = -\frac{11120\times10^6+1440\times10^6}{1191138\times10^4}750-\frac{133030}{34320}+\frac{133030(300+750)}{1191138\times10^4}750 = -15,58\, {N}/{mm^2}$$

$$s_{\mathrm{II}}^{i} = \frac{11120\times10^6+1440\times10^6}{1191138\times10^4}750-\frac{133030}{34320}-\frac{133030(300+750)}{1191138\times10^4}750 = 148,0\, {N}/{mm^2}$$

The final stress state is determined with relations (9):

$$s_c^s = 155,8-29,3 = 126,5\, {N}/{mm^2} < s_{an} = 150,0\, {N}/{mm^2}$$

$$s_c^i = 148,0-75,4 = 72,6\, {N}/{mm^2} < s_{an}$$

4. CONCLUSIONS

If the results obtained through the consolidation methods discussed are analysed, the following conclusions can be drawn:

1. The comparison of this work with “The Consolidation of Steel Bridges Superstructures Without Introducing Initial Stress States” shows that the consolidation methods when an initial stress state is introduced in the structure are more efficient because the steel consumption needed for the consolidation is lower.

2. The comparison between the two methods of the present work shows that the consolidation with pre-stressed steel rod is very efficient, even if the execution is more difficult (the pre-stressing).

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Vertical displacements of a steel-concrete railway superstructure, 51m long, under the 250KN mobile axle load, for speed ranging between 1...150m/s.

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Summary

Objectives:

The paper analyses the vertical displacements of a railway bridge superstructure, steel-concrete composition, 50m span, under the the 250KN mobile axle load.

Work method:

The running track of the bridge has a special structure: the rails are continuously fixed into the concrete slab using the Edilon corkelast material.

In order to determine the impact of the increased speed upon the vibrations and deflections of a mixed section railway bridge superstructure, this superstructure has been carried into the SAP2000 finite element calculation programme.

Sixteen non-linear dynamic analysis have been performed with the 250 KN mobile axle that covers the analyzed model with speeds from 1, 10, 20, 30... 150m/s (3.6...540km/h).

Conclusions:

Based on the results presented in this paper, we can say that, for the high speed trains that run at a speed that is close to 60m/s on this superstructure, it is possible that the vibrations and vertical deflections will be amplified. This amplification will be reached if the frequency with which the axles of the train get to L/2 is close to the own frequency of the first vibration mode of the analyzed structure.

The increase of the running speed of the mobile axle determines the increase of the amplitude of the vibrations, but this does not necessarily lead to a continuous increase of the recorded deflections of the superstructure.

The behaviour of the analyzed model from the point of view of the vibrations and of the deflection pattern corresponds to the known theoretical models.

The paper has a theoretical importance because it studies the behaviour of this type of structure in response to a single mobile axle.

KEYWORDS: superstructure vibration, mobile load, high speed, deflection

1. INTRODUCTION

In order to understand the behaviour of a superstructure under the action of high speed trains, first we need to determine the dynamic answer to the load of one mobile axle. The load with the 250KN mobile axle of the considered model has been chosen in order to be able to make observations regarding the superstructure displacement answer to speeds exceeding the common speeds presently reached on the Romanian railways.

2. OBJECTIVES

To determine the influence of the increased speed of the mobile axle upon the vibrations and deflections recorded in the analyzed superstructure.

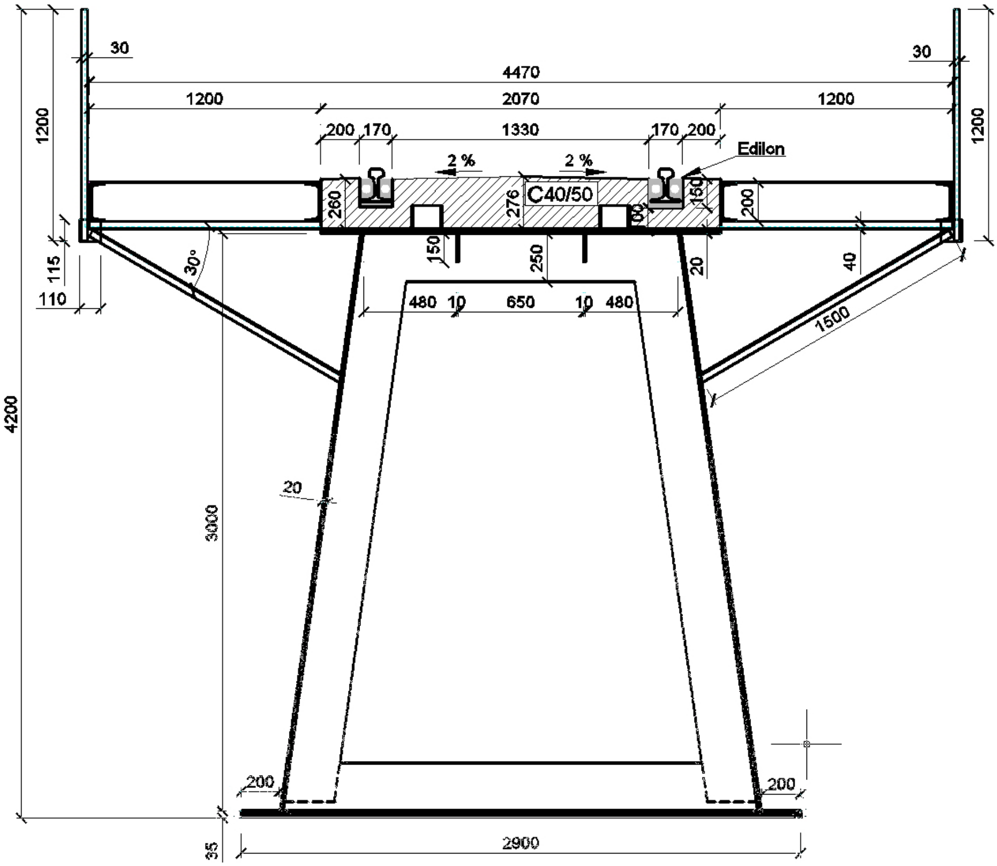

Figure 1. Cross section of the superstructure.

3. WORK METHOD

In order to determine the impact of the increased speed upon the vibrations and deflections of a mixed section railway bridge superstructure, with a 50m span and the cross section as seen in Figure 1, this superstructure has been carried into the SAP2000 finite element calculation programme.

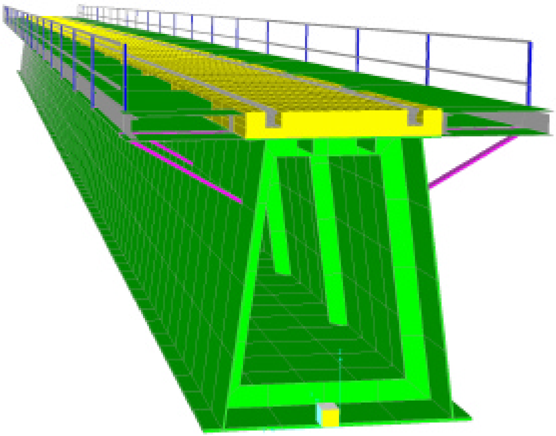

Figure 2. Structure analyzed with the SAP2000 programme

It is important to know the deflections recorded under the action of a mobile axle, in the dynamic analysis for regular or high speed trains, because the second axle of the train will follow a path that has been deformed by the first axle.

In order to obtain results that are as close to reality as possible, we have considered a space moulding of the structure was necessary, with all the comprised elements, instead of making the analysis for a free beam having an equivalent rigidity.

The box section and the sidewalk panes are made of “shell” plane elements, the rails and the linear elements of the sidewalks have been inserted as “frame” rod type elements, the concrete slab the rails are fixed into has been inserted as “solid” type elements.

The analyzed superstructure is 51m-long and it has been divided into 0.5m-long elements along the way, obtaining 103 characteristic sections, as seen in Figure 2. The dynamic analysis have been made considering the P=250KN constant force, charging the structure in 104 load steps. The vertical deflections of the superstructure have been recorded at the rail level, for all of the 104 load steps.

The dynamic analysis have been made using the direct integration method. The vertical deflections obtained in 3 sections, situated at L/4(red), L/2(yellow), and 3L/4(green), are represented in the graphics below.

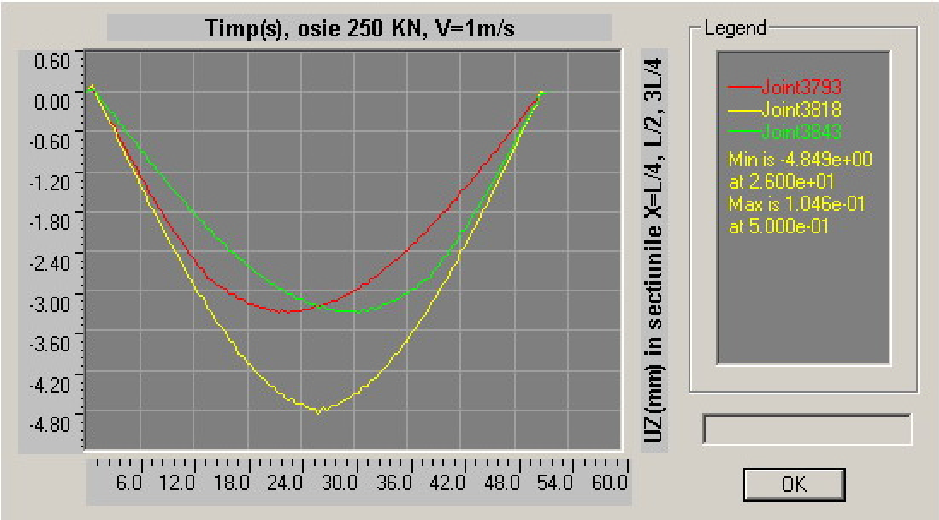

Figure 3. Vertical displacements UZ(mm), P=250KN, V=1m/s (3.6km/h)

In Figure 3, at the speed of 1m/s the vibrations are almost inexistent.

The maximum value of the recorded deflection of the superstructure is 4.849mm and it is reached in the L/2 section, at second 26.

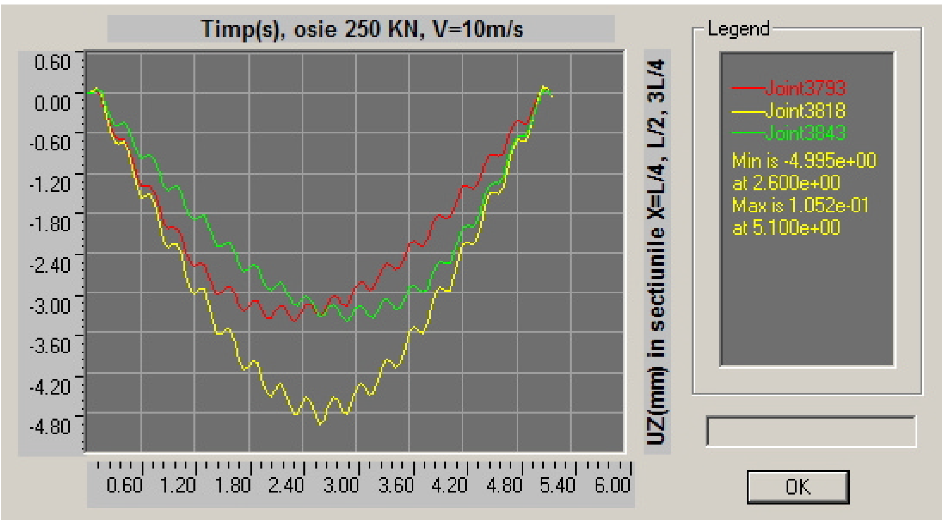

Figure 4. Vertical displacements UZ(mm), P=250KN, V=10m/s (36km/h)

In Figure 4, the speed is 10m/s, vibrations start to appear in the superstructure. The superstructure is loaded for 5.1s, the maximum recorded deflection is 4.995mm.

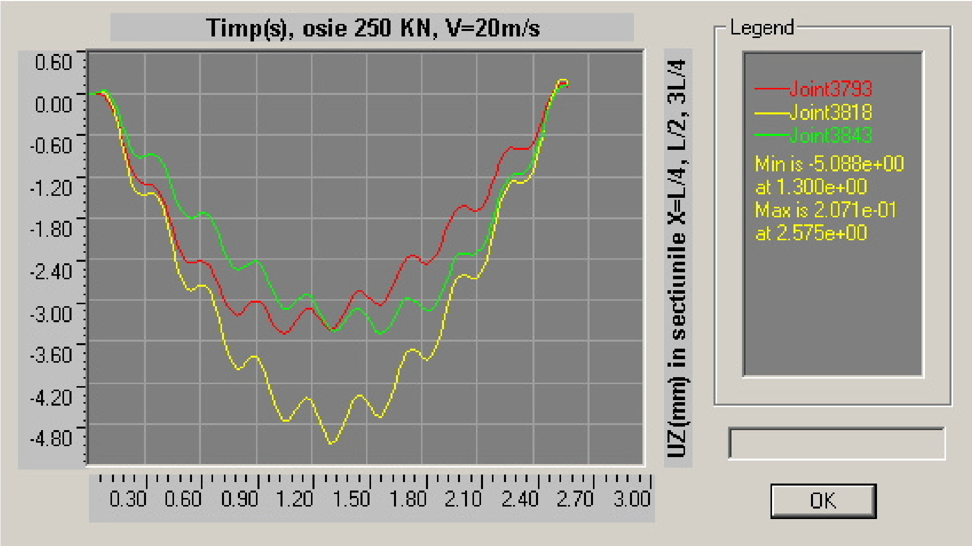

Figure 5. Vertical displacements UZ(mm), P=250KN, V=20m/s (72km/h)

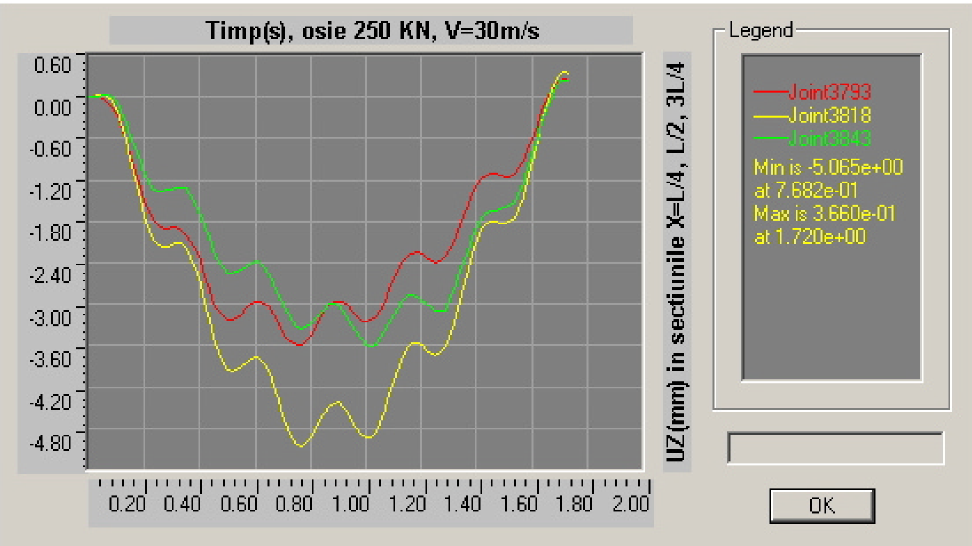

Figure 6. Vertical displacements UZ(mm), P=250KN, V=30m/s (108km/h)

In Figure 5, the 250KN axle will cover the 50m superstructure at 20m/s and the maximum midspan deflection is 5.088 mm.

In Figure 6, the 250KN axle will cover the 50m superstructure at 30m/s and the maximum midspan deflection is 5.065 mm.

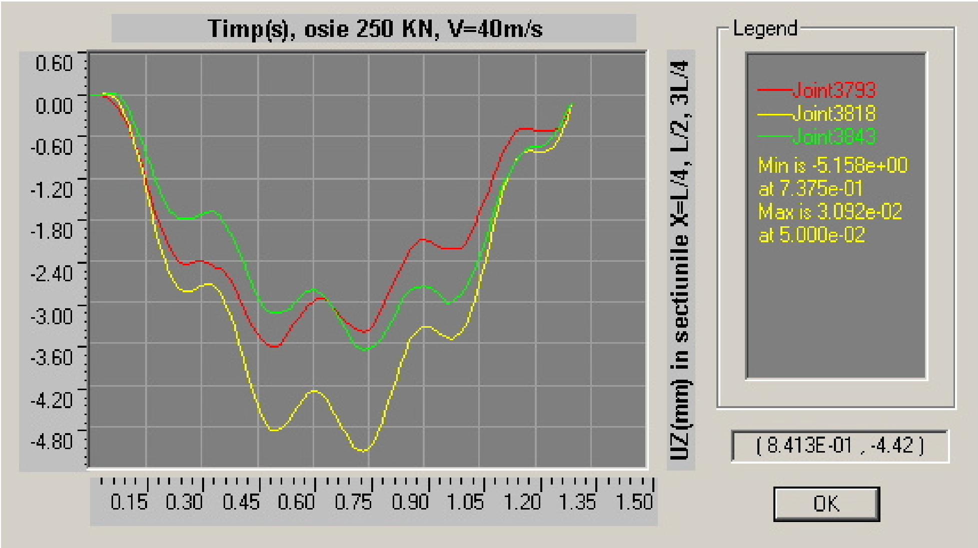

Figure 7. Vertical displacements UZ(mm), P=250KN, V=40m/s (144km/h)

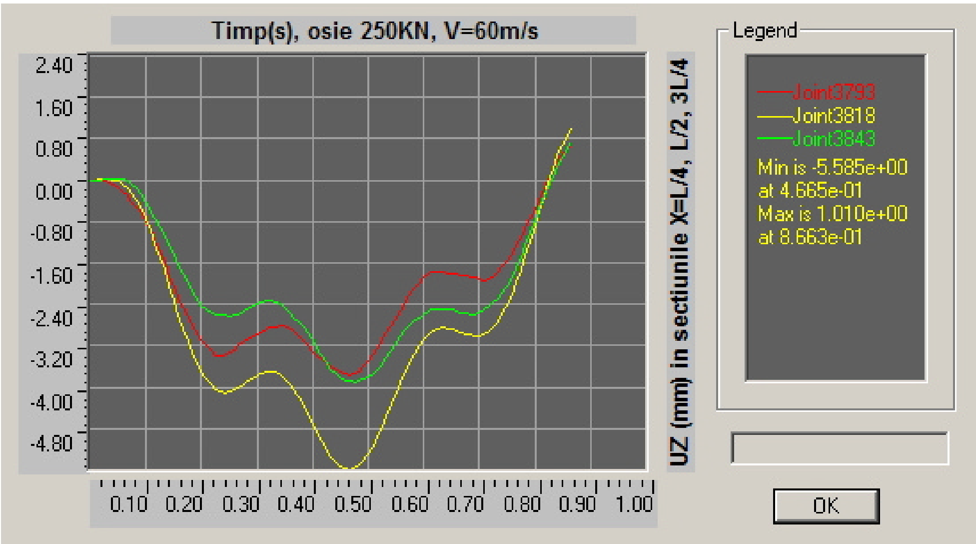

Figure 8. Vertical displacements UZ(mm), P=250KN, V=60m/s (216km/h)

In Figure 7, the 250KN axle will cover the 50m superstructure at 40m/s and the maximum midspan deflection is 5.158 mm.

In Figure 8, the 250KN axle will cover the 50m superstructure at 60m/s and the maximum midspan deflection is 5.585 mm.

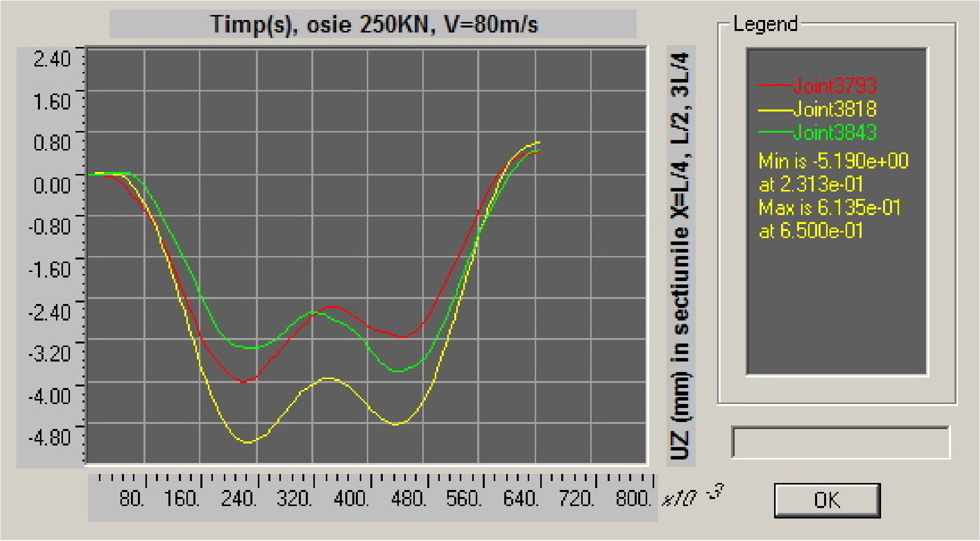

Figure 9. Vertical displacements UZ(mm), P=250KN, V=80m/s (288km/h)

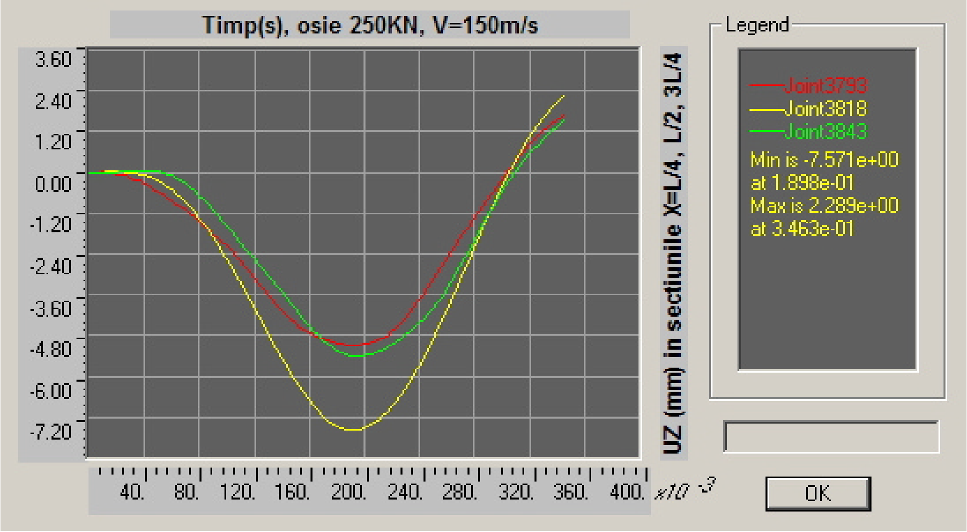

Figure 10. Vertical displacements UZ(mm), P=250KN, V=150m/s (540km/h)

In Figure 9, the 250KN axle will cover the 50m superstructure at 80m/s and the maximum midspan deflection is 5.190 mm.

In Figure 10, the 250KN axle will cover the 50m superstructure at 150m/s and the maximum midspan deflection is 7.571 mm.

4. OBSERVATIONS

The featured graphics allow us to notice that, as the speed increases from 1 to 150 m/s, the time needed to cover the 51m-long superstructure decreases from 51 seconds to 0.34s and the amplitude of the vibrations increases. However, the maximum reached deflections of the superstructure do not compulsorily grow together with the increase of the speed. They are influenced by the vertical position of the mid-span section within the current oscillation. Thus, if the moment the axle gets near the mid-span this section is in a position of minimum of the current oscillation, then the deflection is maximum and it is obtained from static load (that can be considered the maximum value obtained at V=1m/s, where no vibrations have been recorded) to which we add the deflection difference of the current oscillation.

The deflections recorded in the L/4, L/2, and 3L/4 sections normally grow together with the increase of the speed, but there are also speeds for which the maximum deflections obtained are lower than the ones recorded at inferior speeds. The section where the maximum deflections have been obtained is near the mid-span section, but it is not compulsory that it be in this section, as seen in Figure 11.

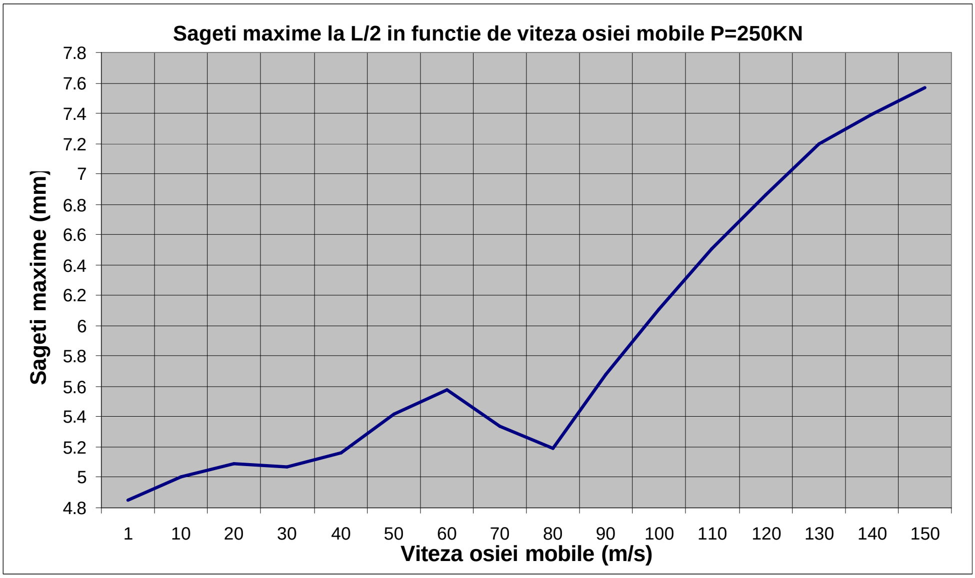

Figure 11. Maximum vertical deflections(mm) recorded at the track level, mid-span of superstructure, for a 250KN mobile axle, speeds V=1…150m/s (3.6…540km/h)

Table 1. is presented below. It features the centralization of the results obtained in the dynamic analysis made with the 250KN mobile axle that covers the 51m-long superstructure, with speeds ranging from 1-150m/s (3.6-540km/h).

Table 1. Maximum deflections recorded at the rail level.

The speed of the mobile axle P=250KN	Maximum deflections UZ (mm) measured in the L/4, L/2, and 3L/4 sections, and the position of the axle on the bridge (the covered distance) at the moment when the maximum value is recorded.						Maximum deflection measured in the rail, the covered distance and the section where the maximum value was recorded.		
V m/s (v km/h)	X=L/4 (13m)		X=L/2 (25.5m)		X=3L/4 (38m)				
	UZ(-) mm	Dist m	UZ(-) mm	Dist m	UZ(-) mm	Dist m	UZ(-) mm	Dist m	Sect m
1m/s (3.6)	3.33	22.5	4.85	26.0	3.32	30.5	4.85	26.0	25.5
10 (36)	3.43	23.0	5.00	26.0	3.43	29.0	5.00	26.0	25.5
20 (72)	3.49	21.0	5.09	26.0	3.48	31.5	5.10	26.5	26.0
30 (108)	3.59	22.5	5.07	23.0	3.61	30.5	5.09	23.0	22.5
40 (144)	3.65	20.0	5.16	29.5	3.70	29.5	5.19	29.0	28.5
50 (180)	3.82	24.0	5.42	24.0	3.62	24.5	5.47	24.5	24.0
60 (216)	3.77	28.0	5.58	28.0	3.89	28.5	5.62	28.0	27.5
70 (252)	3.72	16.0	5.34	31.5	3.97	32.0	5.38	31.5	27.5
80 (288)	4.02	18.0	5.19	18.5	3.81	35.5	5.24	18.0	23.0
90 (324)	4.26	20.0	5.68	20.0	3.71	20.5	5.72	20.0	23.5
100 (360)	4.47	21.5	6.11	21.5	4.04	22.5	6.14	21.5	24.0
110 (396)	4.64	23.0	6.51	23.5	4.36	23.5	6.54	23.5	23.0
120 (432)	4.79	25.0	6.86	25.0	4.65	24.5	6.90	25.0	24.5
130 (468)	4.90	26.5	7.20	26.0	4.93	26.0	7.20	26.0	25.5
140 (504)	5.00	28.0	7.39	27.5	5.18	28.0	7.43	27.5	27.0
150 (540)	5.08	29.5	7.57	28.5	5.42	29.0	7.59	28.5	26.5

5. CONCLUSIONS

Based on the results presented in this paper, we can say that, for the high speed trains that run at a speed that is close to 60m/s on this superstructure, it is possible that the vibrations and vertical deflections will be amplified. This amplification will be reached if the frequency with which the axles of the train get to L/2 is close to the own frequency of the first vibration mode of the analyzed structure.

The maximum deflection in the case of the analyzed superstructure is 7.59mm. This value has been reached for the speed of 150m/s (540km/h) the moment the axle had covered 28.5 of the 51m (it has been recorded in the section placed at 26.5m from the first joint).

The increase of the running speed of the mobile axle determines the increase of the amplitude of the vibrations, but this does not necessarily lead to a continuous increase of the recorded deflections of the superstructure, Figure 11 and Table 1.

The behaviour of the analyzed model from the point of view of the vibrations and of the deflection pattern corresponds to the known theoretical models.
The data offered by this paper are of interest because the second axle of the train will enter the superstructure on a deformed path that records vibrations similar to the ones featured by the graphics Figure 3 – Figure 10, according to the running speed.

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Vertical displacements of a steel-concrete superstructure, 51m long, under the Thalys train load, with speeds ranging between 1...110m/s.

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Summary

Objectives:

To determine the maximum deflections of the analyzed superstructure for the considered running speeds and to determine the critical speed that has as an effect the amplification of the vibrations and the increase of the deflections. The paper studies the behaviour of the superstructure from the point of view of the vertical displacements for the case in which the running track is straight (without counter deflection).

Work method:

The running track of the bridge has a special structure: the rails are continuously fixed into the concrete slab using the Edilon corkelast material.

In order to determine the impact of the increased speed upon the vibrations and deflections of a mixed section railway bridge superstructure, this superstructure has been carried into the SAP2000 finite element calculation programme.

Twelve non-linear dynamic analysis have been performed with the Thalys train that covers the analyzed model with speeds: 1, 10, 20...110m/s (3.6...396km/h).

Conclusions:

The paper analyses the vertical deformations of a railway bridge, steel-concrete composition, 50m span, under the action of a high speed train.

The critical running speed for the analyzed superstructure, train, and speed range is 70m/s (252 km/h). The maximum deflection has been recorded at this speed at midspan of the superstructure; its value was 23.10mm, higher than the 17.61mm deflection recorded at the speed of 110m/s (396km/h). Given that the amplification of the vibrations can appear also at the common running speeds of the high speed trains, we can state that a dynamic calculation similar to the one we have made here is recommended or even compulsory.

KEYWORDS: superstructure vibration, mobile load, high speed train, deflection

1. INTRODUCTION

The increase of the running speed on the railway generates in the bridge superstructures the apparition of certain deflections and vibrations that can be amplified under certain conditions. This paper presents a comparative study of the recorded deflections of a railway superstructure that has a horizontal running track, under dynamic loads (Thalys train), running at speeds between 1...110m/s (3.6...396km/h).

2. OBJECTIVES

To determine the maximum deflections of the analyzed superstructure presented in Figure 1 for the considered running speeds and to determine the critical speed that has as an effect the amplification of the vibrations and the increase of the deflections.

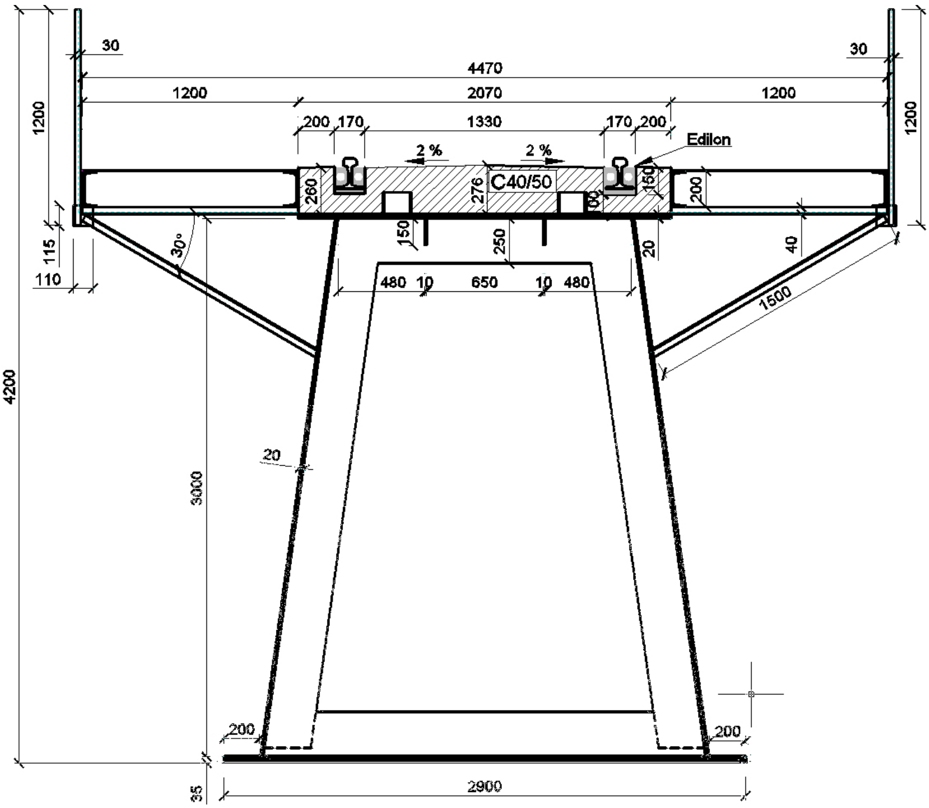

Figure 1. Cross section of the superstructure.

3. WORK METHOD

The superstructure of a mixed section railway bridge without ballast bed, 50m span and the cross section as shown in Figure 1, has been divided and carried into the SAP2000 finite element calculation programme considering elements of 0.5m along the bridge. Thus, 103 characteristic sections have been obtained, and the analyzed model is presented in Figure 2.

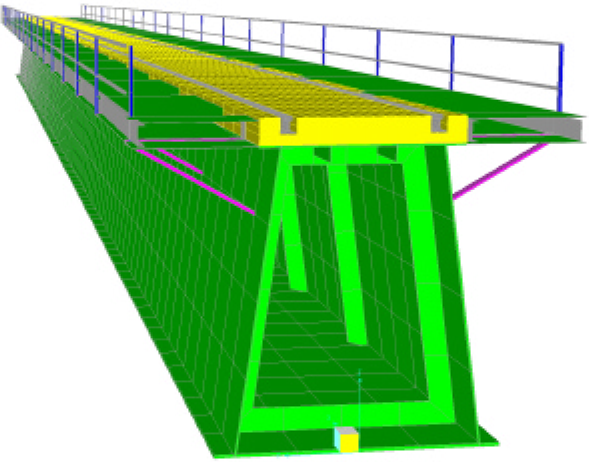

Figure 2. Structure analyzed with the SAP2000 programme

The analyzed superstructure is made up of “shell” plane elements, the rails and the linear elements of the sidewalks have been inserted as “frame” type elements, the concrete slab the rails are fixed into has been inserted as “solid” type elements.

In Figure 3 is presented the high speed train Thalys, made up of 2 engines and 8 intermediary cars, with axle loads between 7.25 and 8.5 tons, the distance between the car bogies of 18.7m. The total length of the train is Lconv=193.14m.

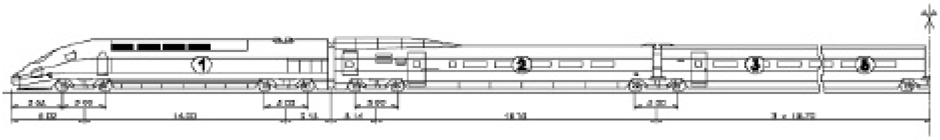

Figure 3. The high speed train Thalys.

From the moment the first axle enters the superstructure until the last axle leaves the superstructure, the train covers 102 elements 0.5m long, resulting 490 successive loading steps. The vertical deflections of the superstructure have been recorded at the rail level, for all the 490 loading steps.

The dynamic analysis are non-linear and they have been made using the direct integration method, with a 5% damping coefficient, directly proportional with the weight. The deflections recorded by the SAP2000 programme in 3 of the 103 characteristic sections, namely the ones situated at L/4(red), L/2(yellow), and 3L/4(green), are represented in the graphics below.

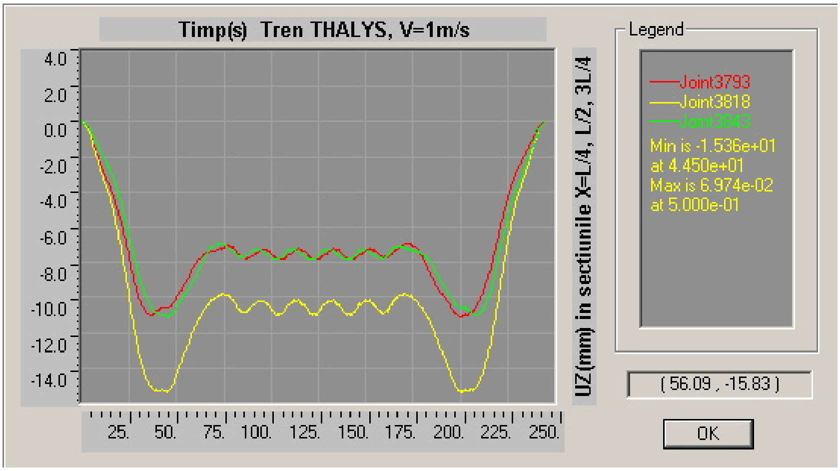

Figure 4. Deflections UZ(mm), Thalys, V=1m/s (3.6km/h)

At the speed of 1m/s, Figure 4, the maximum value of the recorded deflection of the superstructure is 15.36mm. The results are recorded for 245 seconds, the time needed by the train to cover the 51m-long superstructure with the speed V=1m/s. Because of the low speed, the deflection recorded for every loading step can be considered as static deflection for that particular Thalys position on the bridge.

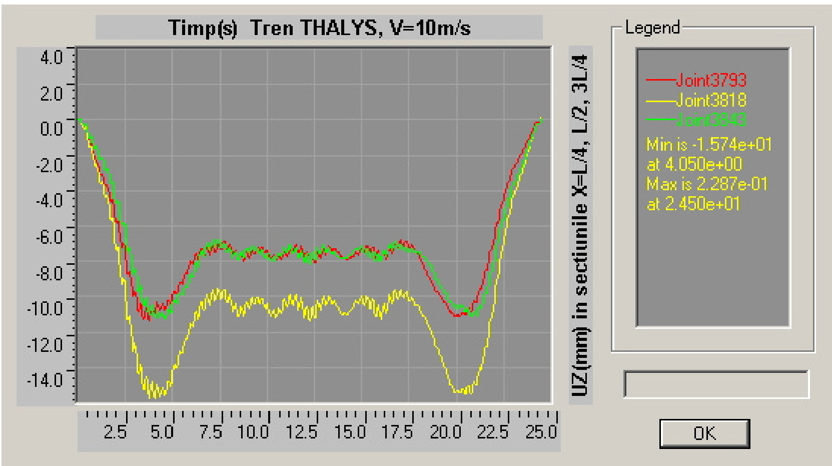

Figure 5. Deflections UZ(mm), Thalys, V=10m/s (36km/h)

When the Thalys speed is 10m/s, Figure 5, vibrations start to appear in the superstructure.The results are recorded for 24.5 seconds, the time needed by Thalys to cover the 51m-long superstructure with the speed V=10m/s. The maximum value of the recorded deflections at L/2 (midspan) is 15.74mm.

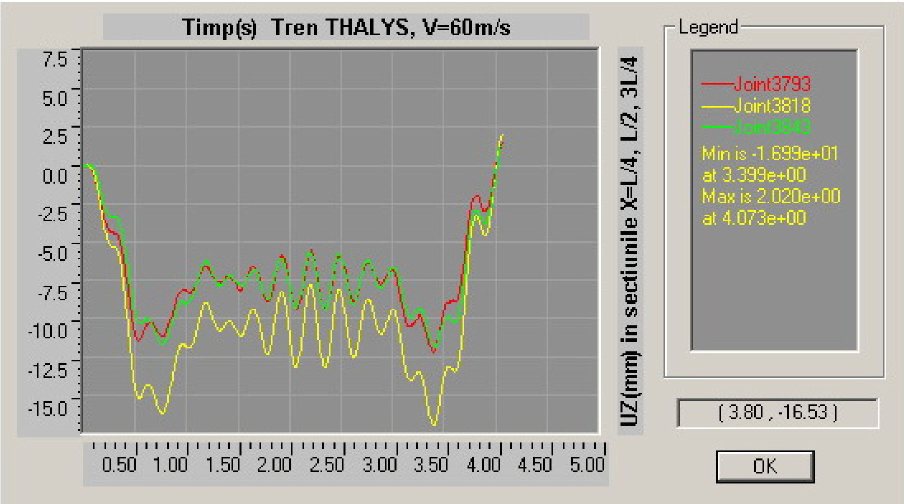

Figure 6. Deflections UZ(mm), Thalys, V=60m/s (216km/h)

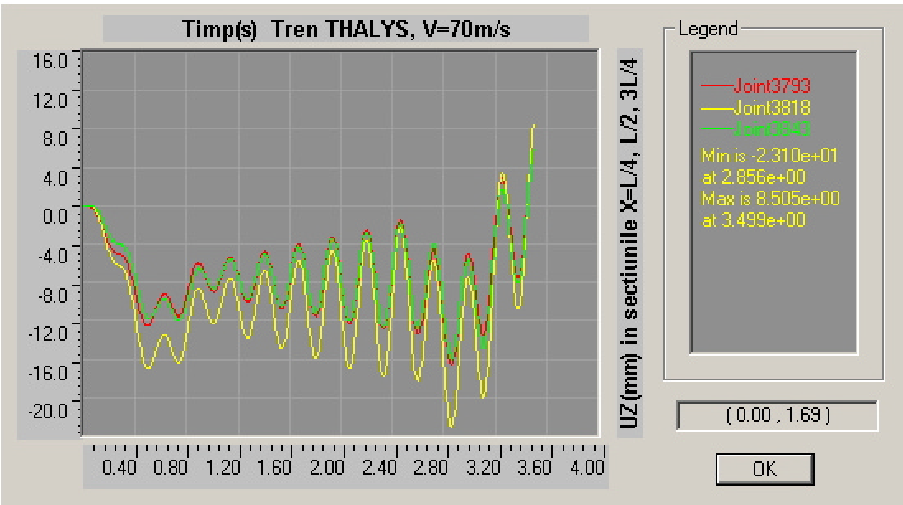

Figure 7. Deflections UZ(mm), Thalys, V=70m/s (252km/h)

At 60m/s, Figure 6, the vibrations amplitude increase, and at 70m/s, Figure 7, will appear the resonance phenomenon. The maximum recorded deflection is 23.10mm.

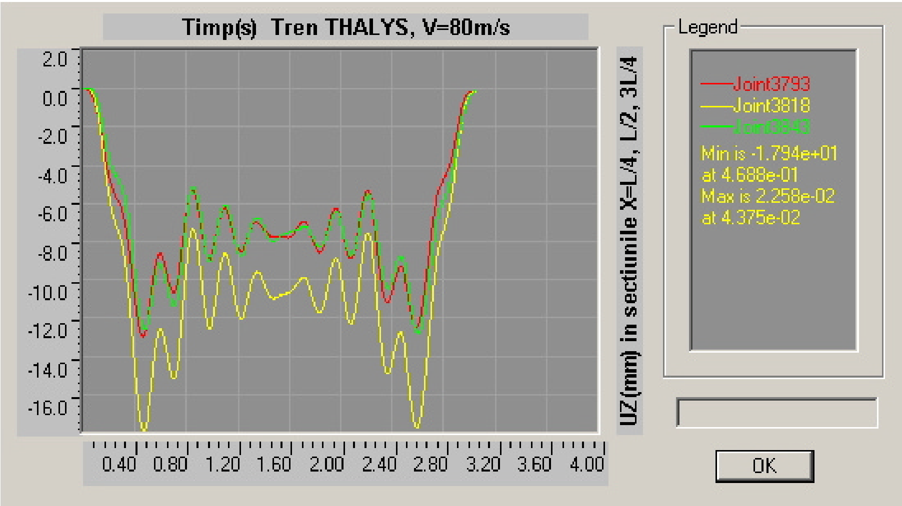

Figure 8. Deflections UZ(mm), Thalys, V=80m/s (288km/h)

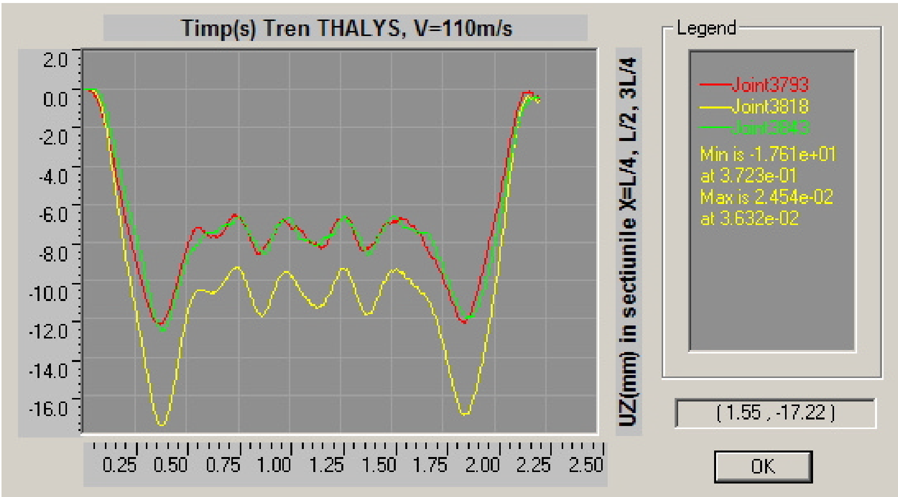

Figure 9. Deflections UZ(mm), Thalys, V=110m/s (396km/h)

When the Thalys speed is 80m/s, Figure 8, the superstructure vibrations amplitude decrease in comparison with 70m/s.

When the loading speed with the Thalys train is 110m/s, Figure 9, the superstructure offer the best high speed behaviour from the vibration point of view.

4. OBSERVATIONS

In the presented graphics we notice that, as the speed increases from 1 to 110m/s, the time needed to cover the 51m-long superstructure decreases from 245s to 2.227s. At the speed of 70m/s (252km/h) a phenomenon of amplification of the vibrations is recorded, the maximum recorded deflection of the superstructure being 23.10mm. At the speed of 110m/s (396km/h) the analyzed superstructure has a much better behaviour, the amplitude of the vibrations decrease, the maximum deflection is 17.61mm.

Table 1 is presented below. It comprises the centralization of the results obtained from the loading of the 51m-long superstructure with a Thalys train.

Table 1. Maximum deflections recorded at the rail level.

THALYS train speed	Maximum and average deflections UZ (mm) measured in the L/4, L/2, and 3L/4 sections						Maximum rail deflections, Afferent section, average maximum rail deflections (mm) throughout the loading time		
V m/s (v km/h)	X=L/4 (13m)		X=L/2 (25.5m)		X=3L/4 (38.0m)				
(0)	UZ(-) max (1)	UZ(-) avg. (2)	UZ(-) max (3)	UZ(-) avg. (4)	UZ(-) max (5)	UZ(-) avg. (6)	UZ(-) max (7)	X (m) (8)	UZ(-) avg. (9)
1m/s (3.6)	11.07	7.25	15.36	10.12	11.06	7.25	15.38	24.5	9.67
10 (36)	11.33	7.25	15.74	10.12	11.23	7.25	15.74	26.0	9.86
20 (72)	11.08	7.25	15.40	10.12	11.12	7.25	15.42	24.5	9.70
30 (108)	11.53	7.23	15.98	10.09	11.40	7.23	15.98	25.0	10.03
40 (144)	11.43	7.25	15.81	10.12	11.41	7.25	15.81	25.0	9.95
50 (180)	11.57	7.25	16.24	10.11	11.51	7.25	16.26	24.5	10.11
60 (216)	12.25	7.25	16.99	10.11	11.86	7.25	16.99	25.0	10.55
70 (252)	16.57	7.28	23.10	10.16	16.04	7.28	23.10	25.0	14.33
80 (288)	12.96	7.25	17.94	10.12	12.83	7.25	17.94	25.0	11.21
90 (324)	13.03	7.25	18.21	10.12	12.90	7.25	18.21	25.5	11.32
100 (360)	13.15	7.23	18.13	10.09	12.83	7.23	18.16	24.5	11.33
110 (396)	12.28	7.26	17.61	10.13	12.62	7.26	17.61	26.0	10.92

The significance of the values in the table:

- for a certain running speed, the SAP2000 programme records in all the 103 characteristic sections of the superstructure all the deflections that occur in the 490 load steps with the Thalys train;

- out of all these sections we have selected the ones from L/2, L/4, and 3L/4 and out of the 490 values recorded in these three sections under loads we have selected only the maximum deflections, inserted in columns (1), (3) and (5);

- for the three analyzed sections we have made the arithmetical average of the 490 values of the deflections recorded in the 490 loading steps (all throughout the time when axles of the train are present on the superstructure) and we have inserted them in the table in columns (2), (4) and (6);

- for each running speed, the highest deflection recorded in the superstructure at the level of the rail is given in column (7), the section where it was recorded is given in column (8), and the average of all the maximum deflections recorded in all the 103 sections is inserted in column (9);

- the sign (-) that appears near Uz means that all the deflections in that column are below the horizontal line of the track in the absence of the train;

5. CONCLUSIONS

The first observation regarding the values in the table refers to the value of the maximum deflections that have been recorded. We notice that the critical running speed for the analyzed superstructure, train, and speed range is 70m/s (252 km/h). The maximum superstructure deflection has been recorded at this speed at midspan; its value was 23.10mm, higher than the 17.61mm deflection recorded at the speed of 110m/s (396km/h). Given that the amplification of the vibrations can appear also at the common running speeds of the high speed trains, we can state that a dynamic calculation similar to the one we have made here is recommended or even compulsory.

Based on a calculation similar to the one we have presented here, the critical speeds that have as an effect the amplification of the vibrations can be pointed out. Afterwards the designer will find through several attempts the critical speed (for this particular case, the attempts with 69m/s, 70m/s, 71m/s, etc., speeds around 70m/s, as shown in the table).

Another observation based on the analysis of the values in this table refers to the variation of the average deflections. This table shows that in the L/4, L/2 and 3L/4 sections, the average values of the deflections recorded in the rail vary insignificantly in relation with the speed of the train (the average values of the recorded deflections in the middle of the superstructure vary between 10.12mm for V=1m/s and 10.13mm for V=110m/s, the maximum value being 10.16mm for V=70m/s).

This fact allows us to state that regardless of the speed of the train, the bridge records vibrations that are close to an average deflection for all the sections of the beam. This value can be considered to be the static deflection recorded in that particular section from the current position of the train.

Based on the average values of the deflections recorded at mid-span with a straight running track, the designer can suggest more shapes of counter deflection to be studied (parabolic, linear side ramps with circular curve in the central area, etc.) so that the maximum value of the designed counter deflection will be around 10...12 mm for this superstructure.

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“Highway and Bridge Engineering 2008”, International Symposium
Iasi, România, December 12, 2008

The Reconstruction Of Beam Superstructures. Case Study – Bearing Devices Replacement For The Independent Deck

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Summary

The present paper consists of two sections. In the first section, the continous beam bridges, located in Moldavia are analysed from two points of view: statistically and as regards the fracture state. In the same time, are underlined reconstruction methods, through modern techniques, which aim at the durable development of these bridge types.

The second section of this paper deals with the replacement technology of the independent deck bearings devices of one bridge located in Gura Vaii, Bacau county . The bridge, built in 1975, with seven spans, with a total length of 196,00m and an 18 metres long independent deck, had faults in the bearing devices area, leading to their replacement.

KEYWORDS: bridge, technical state, static (scheme) model, rehabilitation technology, independent deck (supported deck), bearings.

1. BEAM SUPERSTRUCTURE BRIDGES

1.1. Abstracts

In Romania, up to now, a lot of bridges have been built using a structure called the Gerber beam (suspended and cantilever span girders).This building system, the Gerber beam , has a series of advantages, but in the same time, some disadvantages.

In the last century, experts believed that the use of this building system, for long span bridges, leads to a series of economic benefits. Thus, these benefits refer to low material consumption, if we place the bearing hinges, along the girder, depending on the bending moments distribution. The bending moments distribution, for the Gerber beam, is similar, for the one and the same force system, with the one obtained at a continuous beam (with identical distribution of spans) and with the hinges placed in the sections where the bending moment has zero value.

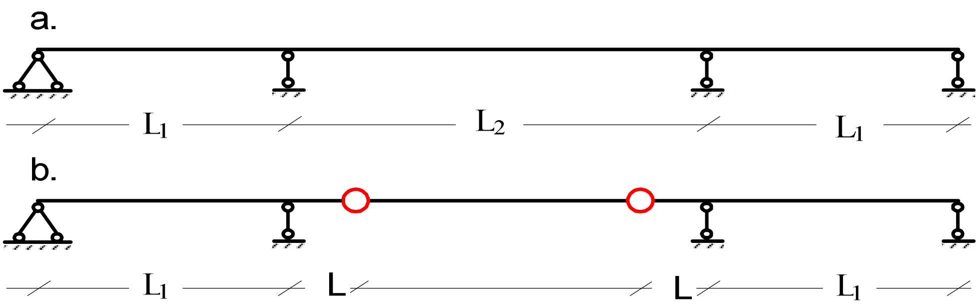
Furthermore, the contingent subsidences and bearings failure, will not introduce supplimentary stress in the Gerber structure, which is statically determined.

The building solutions which involve the use of Gerber beams, have showed some disadvantages too, not only in the erection phase, but also in the exploatation stage.The execution of hinges is a difficult process since it involves a supplimentary reinforcement at the supported beams extremities and at the cantilevers extremities.

1.2. Static Structural Characteristics

The Gerber beams [3], are statically determined, because they consist of several normal beams (cantilever beams, simple beams), which are connected through hinges located in alternate spans.These beams can be considered continous span girders , for which the statistical undetermination has been cancelled through the introduction of the corresponding number of hinges.

In this way, the continuous beam, with three spans from fig. 1. a., two times statistically undetermined, becomes, through the introduction of two hinges at the central span, fig. 1. b., statically determined.



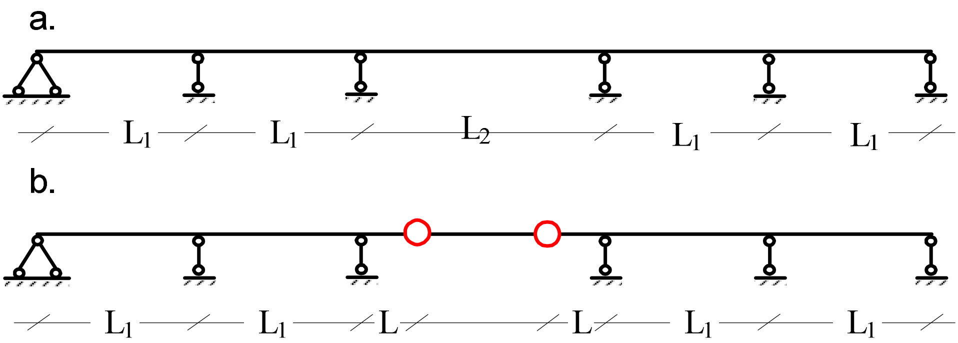


Fig.2. Beams used for bridge superstructures: a. continuous beam; b. continuous beam with hinges

1.3. Examples Of Bridges Executed With Gerber Beams

Over several decades, in the historical area of Moldavia, have been built many bridges with the superstructure on Gerber beams, table number 1. These bridges have been studied in order to determine their *structure* and technical state.

Table 1. Examples Of Bridges with the superstructure on Gerber beams

Crt. No.	Obstacle	Locality	No. of spans	Total length	Year of bridge building
1.	Obrejita river	Obrejita	3	51.0	1977
2.	Milcov river	Golesti	5	137.8	1944
3.	Trotus river	Adjud	9	228.0	1951
4.	Bistrita river	Bacau	7	327	1954
5.	Moldova river	Roman	9	251	1952
6.	Siret river	Siret	5	107.2	1965
7.	Putna river	Vidra	9	230.5	1965
8.	Oituz river	Ferastrau	3	70.01	1961
9.	Oituz river	Oituz	3	68.8	1961
10.	Trotus river	Comanesti	4	132.5	1964
11.	Moldova river	Timisesti	11	289.3	1961
12.	Moldova river	Praxia	9	251.15	1960
13.	Siret river	Gidinti	9	245.25	1966
14.	Jijia river	Cirniceni	7	107.0	1958
15.	Bahlui river	Pd. Iloaiei	3	43.17	1967
16.	Berheci river	Berheci	3	47.2	1960

The next images depict some of the bridges which have been included in the table above.

Photo. 1. Bridge over Moldova river, at Timisesti

Photo 2. Bridge over Berheci river, at Berheci

Photo 3. Bridge over Jijia river, at Cârniceni

Photo 4. Bridge over Oituz river, at Oituz

Photo 5. Bridge over Putna river, at Vidra

Photo 6. Bridge over Putna river, at Comanesti

1.4. Typical bridge failures

During bridge exploitation [1], in the area of the cantilevers, for bridges with a superstructure that has an independent beam, it can be noticed a modification in the shape of the element and in the physical-mechanical properties of the materials.

This general flaw is the result of the accumulation of several failures such as: concrete and the reinforcement corrosion, concrete exfoliation, fractures and cracks etc.

In addition, in case of major flaws in bridge infrastructure, such as pier rotation or the horizontal superstructure displacement, there have been cases where the independent deck fell off the bearings into the river bed.

The photos below depict the technical state of the bearing devices area of the independent bridge deck for the bridges forenamed.

Photo 7. Bridge over Moldova river, at Timisesti

Photo. 8. Bridge over Berheci river, at Berheci

Photo 9. Bridge over Putna river, at Comanesti

Photo 10. Bridge over Jijia river, at Cârniceni

Photo 11. Bridge over Oituz river, at Oituz

Photo 12. Bridge over Putna river, at Vidra

2. CASE STUDY REGARDING BEARING DEVICES REPLACEMENT FOR THE INDEPENDENT DECK OF THE BRIDGE FROM GURA VAII, BACAU COUNTY

2.1. Bridge Superstructure Description

The bridge which is being analysed ensures *the crossing of the 119 county road at 40+066 km.* over the Trotus river in Gura Vaii, Bacau county.

The bridge superstructure is made from *cast- in-place* reinforced concrete beams, by twos in cross section, which from a statical point of view make up a Gerber beam system. The superstructure has a total length of 196,00m and an 18,80m long independent deck. On the S1, S2, S3 and S5, S6, S7 spans, the deck is continuous, and the S4 span (central) is made from an independent deck.

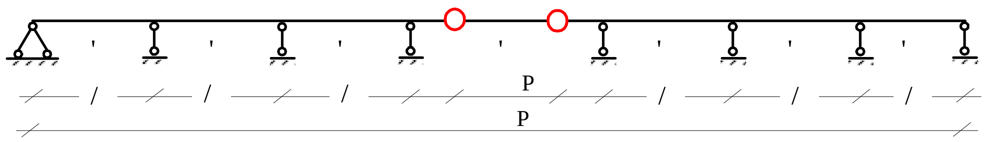


Fig. 3 Static Scheme of the Bridge Superstructure

The bridge superstructure *ensures* a *carriageway* with a width of 7,00m and two sidewalks with a width of 1,50 m each, therefore, the total width is 10,60m. The roadway and sidewalk coating is made from asphalt mixtures.

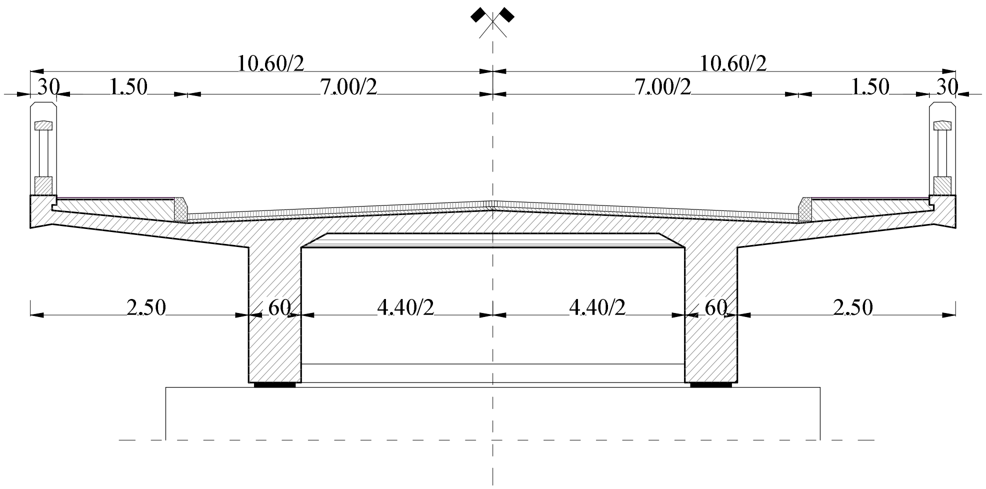


Fig. 4. Superstructure Cross Section

The infrastructure consists of two abutments and six piers. The abutments elevations are from concrete with elastic elements from reinforced concrete (abutment wall, back walls) and concrete direct foundations

The transition from the access ramps to the bridge structure is made by cone quarters.

The piers elevations are from reinforced concrete with concrete direct foundations.

2.2. Degradation State

The bridge shows damage of the central span beams (independent beam, beared in Gerber system on the cantilevers of the side continous beams): strong weeping through the joints from the independent beam ends, degraded concrete at the level of the joints and degraded bearing devices. The reparation calls for the lifting of the independent deck –the replacement of the bearing devices and the reconstruction of the concrete from the cantilevers area [4],[5].

Photo 13. Concrete degradation in the bearings area

Photo 14. Degradations of the mobile-bearing devices

Photo 15. Degradations of the fixed-bearing devices

Photo 16. Degradations of the fixed-bearing devices (detected after the lifting of the deck)

Photo 17. Degradations of the concrete in the mobile-bearing devices area, degradations of the bearing device elements and the blockage of the construction joint by aggregates from the road coating.

2.3. Typical Devices and Equipment for the Replacement of the Independent Deck Bearing Devices

? The Steel lifting beams system, consisting of steel beams-supporting beams-anchor tie-rods; for the lifting of the independent deck were used four steel beam systems operated simultaneously by four hydraulic cylinders of 200 tons capacity each;

Photo 18. Lifting Beams Structure (top view)

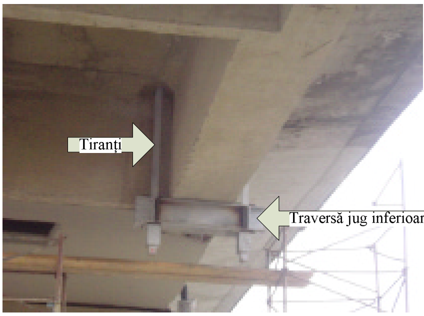


Photo 19. Lifting Beams Structure (bottom view)

? Hydraulic Power Tools

- o four hydraulic cylinders of 200 tons capacity each;

Photo 20. Hydraulic cylinders with double action

- Two hydraulic drive units - 700 bar capacity:

Rezervor	10; 20; 30; 40 l
Debit presiune înaltă	0,5 – 1,5 l/min
Debit presiune joasă	6 - 16 l/min
Motor electric acționare	1,5 - 4 kW
Presiunea maximă	700 bar
Afișarea digitală a presiunii	
Reglaj proporțional al presiunii	
Înregistrarea presiunii	
Masa	65 - 160 kg

Photo 21. Hydraulic drive unit

- Hydraulic drive units fittings:

Photo 22. Hydraulic drive unit fittings: a) Quick high voltage coupler; b) hydraulic hose; c) branch hydraulic systems

? Steel supports which ensure the bearing of the lifting beams:

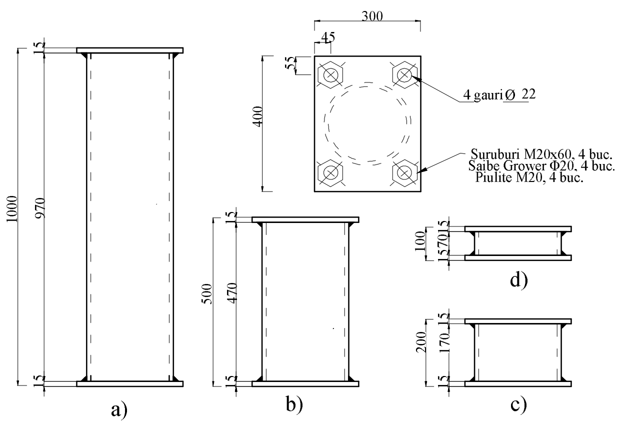


Fig. 5. Steel supports: a) h=100cm (8 pieces); b) h=50cm (8 pieces); c) h=20cm (16 pieces); d) h=10cm (16 pieces);

2.4. Deck Lifting Technology

The independent deck lifting was done through the medium of a four steel beam system (photo 18,19), using four hydraulic cylinders of 200 tons capacity each (photo 20) operated simultaneously by two hydraulic drive units (photo 21)

The purpose of the operation was the deck lifting to a ? h=1,30m height, in order to ensure the access for the bearing devices replacement and for the redressing of the degraded concrete surfaces from the bearing devices area (the covered surfaces of end crossbars, the inner surface of the supported beams).

The independent deck lifting involves the following stages:

? The preparatory stage:

- the pickling of the asphaltic layer and of the equalization concrete from the supports and hydraulic cylinders area;

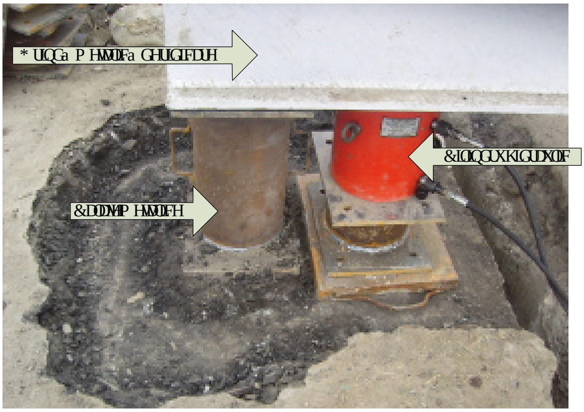


Photo 23. The preparation of the steel supports and hydraulic cylinders bearing area.

- the marking and execution of the holes in the deck for the anchor tie-rods;

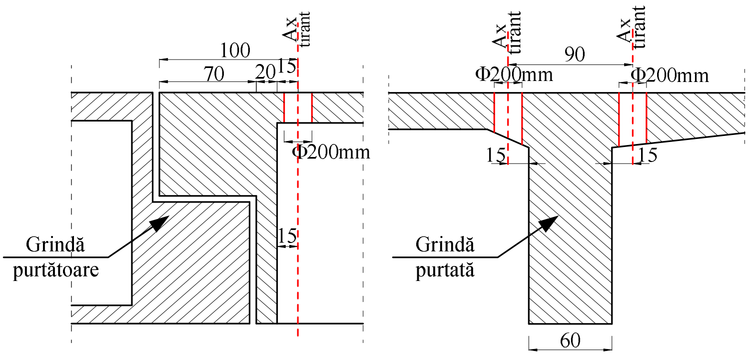


Fig. 6 The Marking of the holes for the anchor tie-rods

Photo 24. The execution of the holes for the anchor tie-rods

- the disposing of the covering devices of the joints on the carriageway and the sidewalks area;

Photo 25. The disposing of the covering devices of the expansion joints

- The erection of the lifting device;

Photo 26. The installation of the lifting device

- the installation of the hydraulic and electrical equipment;

Photo 27. The installation of the hydraulic and electrical equipment

? The lifting of the independent deck

- the lifting of the deck using four hydraulic presses operated simultaneously on two hydraulic drive units. For the temporary bearing of the lifting beam were used steel supports (h=100 - 1000mm) and hard wood supports (h=50mm);

Photo 28. The deck lifted with ? h=40cm

The independent deck lifting was done, depending on the maximum hydraulic cylinders stroke, in steps of 10cm; which resulted in the steel supports modulation with heights ranging between 10 -100cm.

- The cross- bracing of the steel supports in intermediate stages – horizontal bracings on the lower side;

Photo 29. Horizontal bracing on the lower side of the steel supports

- Dupa ridicarea tablierului la înaltimea ? h= 1,10m, se monteaza contravântuiri verticale si orizontale la partea superioara a calajelor ; After the deck has been lifted to the ? h=1,10m height, horizontal and vertical bracings are mounted on the upper part of the steel supports;

Photo 30. The cross-bracing of the steel supports

- In the final stage, after the deck was lifted to the ? h=1,30m level, the hydraulic cylinders were removed and replaced by steel supports wedged up with hard wood supports in order to ensure an accurate bearing of the stiff beams lengthwise the bridge;

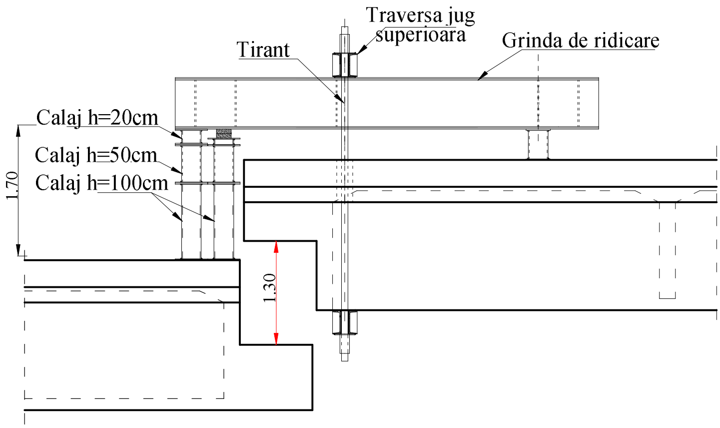


Fig. 7. Deck lifted to the ? h=1,30m level

Photo 31. The final bearing of the lifting beam

After the repairing of the bridge superstructure, the stage of descending the deck on the new bearing devices was started, with the following operations sequence:

? installing of the hydraulic and electrical equipment;
? disassembling of the horizontal and vertical bracings of the steel supports;
? descending the deck on the new bearing devices;
? disassembling of the lifting device – the steel beams - supporting beams - anchor tie-rods system;

All these construction works took place under the following conditions: vehicle traffic was closed and the pedestrian traffic was supervised by a beneficiary's representative.

In selecting this technology of independent deck lifting, was used a feasibility study which took into consideration two lifting procedures:

? The lifting of the deck from *temporary steel piers* located under the central span – a solution which involved more building works and more metallic material, and obviously higher costs; the advantage of this solution would be a lower degree of complexity and a stiffer deck bearing;

? The lifting of the deck using the steel beams system which was presented above – a solution which involved fewer building works and less metallic material, lower costs; this solution involved a higher degree of complexity from the point of view of ensuring the durability of the structure – flexible bearing with potential large displacements;

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Corrosion Durability of Recycled Steel Fibre Reinforced Concrete

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Abstract

Steel fibre reinforced concrete (SFRC) is known by its excellent performance when compared to conventional concrete. The use of steel fibres in concrete may contribute to improve properties such as crack and impact resistance, shrinkage reduction and toughness, by preventing/delaying crack propagation from micro-cracks to macro-cracks. In some cases it may be used as a replacement to conventional steel reinforcement or to high quality aggregates in roller compacted concrete.

Industrial steel fibres can be found in a wide range of types, aspect ratios and properties. However, steel fibres are dispersed in all directions and for the same flexural capacity a larger volume of fibre is required than for conventional reinforcement, thus increasing costs. An alternative to industrial fibres comes from the recycling industry through the use of steel from post-consumer tyres. Even though fibres reclaimed in this process are not uniform and have variability in size, experiments at the University of Sheffield have demonstrated that they can be used to enhance the mechanical properties of concrete. In addition to enabling a good mechanical behaviour, these fibres could become an interesting reinforcing alternative due to their lower cost and associated environmental benefits.

Since the concrete produced with recycled fibres is a new material, the study of its durability becomes mandatory before applying it to large scale structures. This research aims to provide means for better understanding of the deteriorative processes that may contribute to the performance reduction of SFRC with recycled fibres. For that, fatigue process, shrinkage, freeze-thaw and corrosion of these fibres are discussed. For the latter property, a complete experimental plan has been already carried out by accelerating corrosion by wet-dry cycles. The results in terms of visual observations show that corrosion affects negatively the appearance of SFRC specimens. The mechanical properties, through the analysis of flexural and compressive tests, are not affected by corrosion after 5 months of wet-dry cycles.

KEY-WORDS: RCC, Steel fibres, corrosion, durability

1 INTRODUCTION

When steel fibres are added to concrete, the possibility of corrosion of the fibres is a durability parameter that requires special attention. Corrosion is a very common pathological manifestation occurring in ordinary reinforced concrete (RC), and also one which causes more damage to structures, and thus, analysing the effects of this phenomenon in SFRC is of great importance. Several studies have already been carried out considering the corrosion on SFRC (Mangat and Gurusamy, 1987 and 1988; Granju and Balouch, 2005; Kosa and Naaman, 1990). However, the conclusions presented in these works are not sufficient to attest the corrosion resistance of SFRC.

Besides industrial fibres, this research will also analyse the use of recycled steel fibres, produced from post-consumer tyres. This type of fibres can be used as an alternative to the high cost of industrial fibres and also as an environmentally friendly material. Since the concrete with these fibres is considered as a new material, no studies can be found in the literature. The same can be extended to practical applications, on which no durability resistance has been attested so far.

In ordinary RC, steel bars are protected by the passive oxide layer due to the high pH of the concrete and the presence of calcium-hydroxide. In SFRC, however, fibres are randomly distributed inside concrete. This means that some fibres are not protected by the alkalinity of concrete because their concrete cover is near zero (Mangat and Gurusamy, 1988). There are three hypotheses to explain the consequences of corrosion in SFRC. In the first, a decrease of peak loads and an embrittlement of the post peak behaviour is expected to occur; in the second, it is assumed that rust formation will increase friction between the fibre and the cement matrix, which can lead to an uniform gain in strength; and finally, the third hypothesis is related to autohealing of cracks, which restores concrete continuity through the crack and increases the peak load (Granju and Balouch, 2005). The first hypothesis is also attested by Nordstrom (2000) who says that structures with relatively thin steel fibres may have the load capacity reduced due to a decrease of the fibre diameter. The author also adds that this reduction in performance may be highlighted in cracked specimens.

Nordstrom (2000) cites that steel fibres may corrode at a lower rate than conventional reinforcement when both are placed in the same conditions. It is also assumed that the rate of degradation in SFRC may not be linear.

The focus of this paper is to present the study, developed to evaluate the corrosion durability of concrete for pavement applications. For that, ordinary wet mixes were studied as well as dry mixes compacted by rollers. The latest one (RCC) is a dry Portland cement concrete which has zero-slump. In pavement application, which is the focus of the research reported in this paper, RCC is placed by using asphalt pavers, equipped with heavy duty dual tamping screed, and compacted by rollers.

The majority of studies currently undertaken on steel fibre reinforced (SFR)-RCC are focused on investigating of the mechanical properties. Steel fibres in RCC may be used as a replacement for high quality materials and may also enhance RCC's weak properties, such as low performance against fatigue and tensile stresses and low resistance against crack propagation.

1.1 Procedures to accelerate corrosion in SFRC specimens

Corrosion is usually considered as a long-term process. The effects of the corrosion phenomenon in real structures are dependent of the aggressivity level of the environment, the concrete quality and the maintenance procedures, if they exist. For this reason, techniques to accelerate corrosion in concrete specimens are often used in research centres. Several studies can be found on the techniques to accelerate corrosion in ordinary RC specimens (El Maaddawy and Soudki, 2003). However, there is a lack of studies and standards considering the best way to accelerate corrosion in SFRC.

Some studies found out that the best way to undertake corrosion tests is by using a salt-spray chamber (Mangat and Gurusamy, 1987 and 1988), while others believe that dry-wet cycles is the best technique (Granju and Balouch, 2005 and Kosa and Naaman, 1990), and finally, some of them prefer to expose the specimens directly to a real marine environment (Mangat and Gurusamy, 1988). Electrochemical tests usually used to accelerate corrosion in ordinary RC are not recommended for SFRC. Steel fibres may provide a continuous electrical path between the edges of the specimen. This will lead to fibres more corroded than others, thus invalidating the test. Even though there are some studies on accelerated corrosion techniques of SFRC, the number of researches is still insufficient to attest the best technique to do it.

2 EXPERIMENTAL PROGRAMME

An experimental plan has been carried out to measure the losses in terms of mechanical properties in SFRC and SFR-RCC specimens due to corrosion. For that, prisms (150x150x550mm) and cubes (150mm) were cast to evaluate the performance against flexural and compressive strength tests, respectively. A visual investigation was also performed by analysing the external and internal surfaces of the specimens.

2.1 Mix proportion

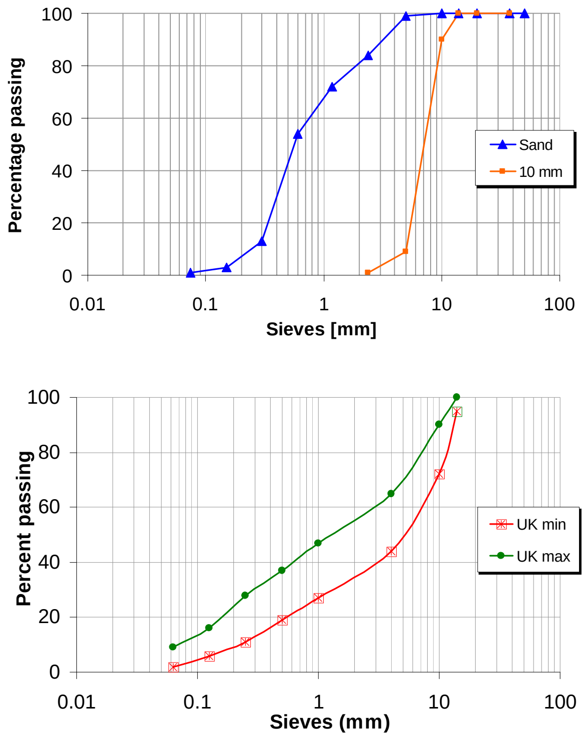
For the purpose of this research, SFRC and SFR-RCC specimens were cast following the concrete mix proportions shown in Table 1. The mixes have already been studied in previous experiments carried out in The University of Sheffield (Angelakopoulos et al., 2008) on which the main purpose was to investigate concrete pavement applications. The main difference between the mixes presented in the table is the amount and type of steel fibre. Plain concrete (without any fibre addition) was also cast to quantify the improvement in the performance caused by both fibre types. The amount of industrial fibres was chosen considering that 2% (by mass) is the average content used in practical applications. The recycled fibre content (6% by mass) was chosen because this is the possible amount which gives approximately the same flexural performance as 2% industrial fibres.

Table 1 – SFRC and SFR-RCC mix proportion per cube meter of concrete.

Mix	Cem [kg]	Sand [kg]	Aggregate [kg]	Water [kg]	Super plastic [%]	Air entrainer [%]	Industrial fibres [% by mass - kg/m³]	Recycled fibres [% by mass - kg/m³]
SFRC1	380	833	1004	133	0.7%	0.135%	-	-
SFRC2	380	833	1004	133	0.9%	0.135%	2% - 47	-
SFRC3	380	833	1004	133	1.3%	0.135%	-	6% - 141
SFR-RCC1	300	-	2092	160.3	-	-	-	-
SFR-RCC2	300	-	2092	160.3	-	-	2% - 51	-
SFR-RCC3	300	-	2092	160.3	-	-	-	6% - 153

Cement used for wet mixes is a low energy cement (LEC), which consists of granulated blast furnace slag, secondary constituents, calcium sulphate and additives. Corrosion protection is due to the pH around 11.8 to 11.9. For RCC mixes, the cement used was Type II - limestone, which has a corrosion protection slightly higher than the LEC, through the pH around 13.

The gradation curve for the wet mix round aggregate 10mm and RCC crushed aggregate blend 14mm is presented in the figure below.



a)

b)

Figure 1 – Gradation curve for the coarse aggregate. a) wet mixes; b) RCC mixes

The industrial fibre used, termed I2C1/54, had a cone at each end (Figure 2a), an aspect ratio of 1/54 and tensile strength of 1100N/m². Recycled fibres were obtained from post-consumer tyres, through mechanical treatment and 90% of the fibres had length in the range of 3-11 mm. Figure 2b illustrates these fibres.

Specimens were cast in steel moulds and compacted by an external vibrating machine (wet mixes) or hydraulic hammer (RCC mixes). All specimens were demoulded 24h after casting and then placed for 27 days in the mist room with controlled temperature and relative humidity (+20°C and RH=95%). After this period, specimens were subjected to wet-dry cycles in order to accelerate corrosion in the fibres, as explained in more detail in the next section.

a) industrial fibres

b) recycled fibres

Figure 2 – Appearance of fibres.

2.2 Acceleration of corrosion

As mentioned before, there is no standard on the best technique to accelerate corrosion in SFRC specimens. Furthermore, all studies carried out in the area show different techniques and no correlation among them is presented.

For the purpose of this research, the procedures used to accelerate corrosion were by immersing RCC specimens in a container with chloride solution (3% of NaCl) for 4 days followed by a period of 3 days of drying at ambient temperature. Wet-dry cycles allow both the penetration of chlorides and the presence of oxygen, in order to develop the corrosive process. Figure 3 shows the wet and dry phases of the test.

a) specimens immersed in saline solution – 4 days

b) specimens drying at ambient temperature – 3 days

c) return to salt solution – end of cycle

Figure 3 – Steps of the wet-dry cycles – acceleration of corrosion.

Plastic spacers were used to separate the specimens from each other (by at least 10mm). The procedures to take the specimens out and in the containers were performed mechanically by an overhead crane (see above figure) which resulted in less than 5 minutes the time spent in each procedure. These experiments are considered as long-term tests and are expected to include 5 and 10 months of continuous cycles.

3 ANALYSIS OF THE RESULTS

This section presents the main results regarding the losses in performance due to corrosion attack after 5 months of continuous wet-dry cycles. The results after 10 months of corrosion acceleration will be obtained in autumn 2008.

3.1 Visual analysis

A series of pictures have been taken to visualize the various stages of corrosion testing before starting the corrosion cycles (Figure 4); at 2 months and half (Figure 5), and finally, at 5 months (Figure 6). For the latter, following the flexural and compressive strength testing of the specimens (at month 5), a visual examination was carried out (Figure 7) to asses the internal effects of corrosion (i.e. inside the samples).

Figure 4 shows the specimens just after being taken out from the mist room.

a) specimens being positioned in the frame before corrosion

b) specimens ready for the beginning of cycles

Figure 4 – Specimens before corrosion accelerating procedures.

It is clearly observed in the pictures that, during curing, some of the specimens have already been externally corroded. This is especially noticed for the RCC prisms reinforced with recycled fibres. Despite this fact, all other samples seem to present no signs of corrosion.

a) overall view of the samples b) view of corroded RCC sample with recycled fibres c) view of corroded RCC sample with industrial fibres

Figure 5 – Specimens after 2.5 months of corrosion accelerating procedures – external analysis.

Figure above shows the specimens when taken out from the solution after 2.5 months of accelerated corrosion. It is noticed that the samples with more superficial corrosion effects are those already corroded after curing (RCC with recycled fibres). A large amount of rust is observed in Figure 5b, which changed completely the specimen’s colour. In this same set of specimens, spalling of small parts of concrete occurred by just using little effort. Figure 5c shows industrial fibres with a great amount of corrosion. This is especially observed in the fibres which were already exposed to the surface of the RCC specimens due to compaction limitations during casting. These specimens are not as rusty as those with recycled fibres. Wet mixes did not present as high corrosion signs as the RCC mixes.

a) overall view of the samples b) view of corroded RCC sample with recycled fibres c) view of corroded RCC sample with industrial fibres

Figure 6 – Specimens after 5 months of corrosion accelerating procedures – external analysis.

Figure 6 shows that after 5 months of corrosion acceleration, the same comments explained for 2.5 months apply regarding the external appearance of the specimens. The level of the superficial damage, however, has increased from the previous assessment. Wet mixes presented only few signs of external rust, usually on the fibres positioned near the specimen surfaces.

The external effects of corrosion produce a brownish colour all around the specimens. This appearance, when noticed in real structures, may be responsible for uncomfortable feelings in users.

An observation of the internal appearance of the concrete after testing the specimens was also performed. The purpose of this was to verify if the salt solution is able to penetrate inside concrete, thus causing corrosion of fibres in different depths of the specimens (Figure 7).

a) RCC sample with recycled fibres after compressive test

b) detail of inside sample's edge – little recycled fibres corroded

c) detail of inside sample's edge – industrial fibres partially corroded

Figure 7 – Specimens after 5 months of corrosion accelerating procedures – internal analysis.

Figure 7a shows that the effects of corrosion, such as rusty appearance, can be seen only externally. Figure 7b, for instance, shows that just a small layer (around 10mm) inside the sample presented few signs of corrosion (for RCC samples with recycled fibres). Figure 7c shows the edge of a RCC sample with industrial fibres; corrosion took place only in the fibre surface exposed to the environment. In the same fibres, the surface anchored to concrete presented no corrosion, as it was protected by the alkalinity of concrete. Wet mixes did not present any sign of internal corrosion.

3.2 Flexural strength

The test was performed according to the RILEM TC 162-TDF (2002) by controlling the crack mouth opening displacement in a four-point flexural test.

Specimens were notched transversally in the bottom centre (25mm depth) to stimulate the crack to open in that region. Figure 8 shows a specimen during testing.

Figure 8 – Specimen in the flexural test.

The results in terms of flexural strength obtained from the peak loads are presented Figure 9.

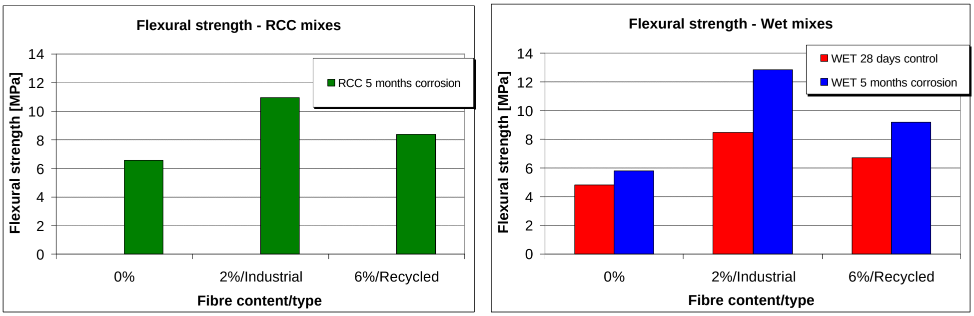


Figure 9 – Flexural strength results for RCC and wet mixes.

It can be observed that for the wet mixes there is an increase of the flexural strength capacity of the samples after 5 months of corrosion when compared to the ones tested at 28 days. This increase is possible caused by the difference in the age of the specimens. Results for RCC control specimens for 28 days will be available by autumn 2008. However, it can be observed that even when corroded, the RCC specimens with fibres presented better performance when compared to the plain concrete.

Figure 10 shows bending load versus vertical displacement results (average of two LVDT positioned at each side of the specimens). By considering typical results of flexural test of SFRC, it seems that there are no changes on the post crack behaviour of a corroded SFRC specimen compared to a non- corroded one.

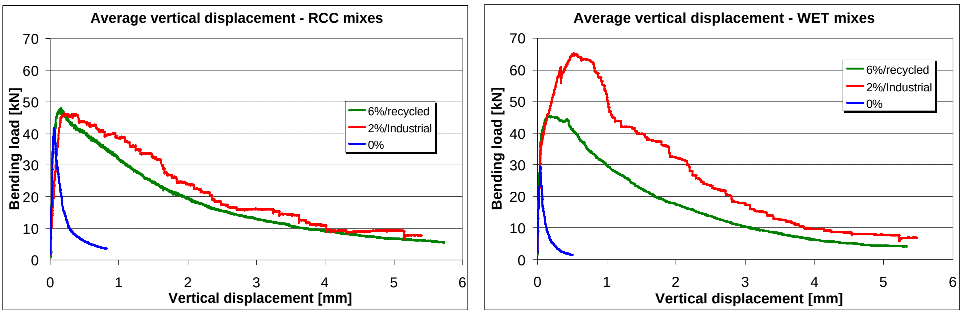


Figure 10 – Flexural test results after 5 months of corrosion accelerating procedures.

3.3 Compressive strength

Compressive strength tests were performed according to the British Standard BS EN 12390-3 (2002). Specimens were tested after being subjected to 5 months of continuous wet-dry cycles. The results obtained are presented in Figure 11.

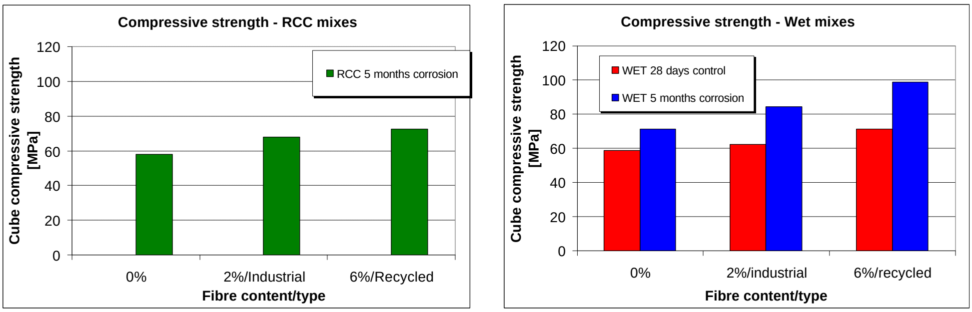


Figure 11 – Compressive strength results for RCC and wet mixes.

Following the same behaviour as the flexural test, it can be observed that samples subjected to 5 months of corrosion presented better results when compared to the control samples (wet mixes at 28 days). Same comments regarding the missing control samples (28 days) for the RCC mixes in the flexural strength test is applied to this case (results will be available in autumn 2008).

Industrial fibres present better performance than recycled fibres in the flexural test. However, recycled fibres showed high compressive capacity than industrial fibres.

4 DISCUSSIONS AND CONCLUSIONS

The following items explain the main conclusions obtained from this work.

The wet-dry procedures used to accelerate corrosion in SFRC can be considered as a good technique to imply corrosion in concrete specimens in a small period of time.

In terms of external analysis, specimens reinforced with industrial fibres presented less damage than the ones with recycled fibres. The internal analysis showed that both fibre types presented limited signs of corrosion. RCC specimens seem to develop more external rust.

Considering the flexural behaviour of corrosion SFRC, it was observed that the performance of specimens was not reduced by corrosion attack. The same conclusion can be extended to compressive results, which showed that, even if specimens are externally corroded, this has no influence on the compressive strength capacity of the samples. The increase in the capacity observed for corroded wet mixes when compared to those at 28 days is probably due to the age of the specimen.

Compressive and flexural results presented in this paper showed that specimens with 2% industrial fibres have similar performance as those with 6% recycled fibres. Industrial fibres presented better performance in flexural tests while the opposite was noticed in the compressive test.

The overall analysis of the results demonstrated that when SFRC specimens are subjected to 5 months of continuous wet-dry cycles, corrosion of fibres can be noticed only externally. Just few signs of corrosion can be visualised internally and both external and internal visual effects do not interfere on the mechanical capacity of the samples. When used in pavements, if SFRC and SFR-RCC is covered with asphalt layer, the possible uncomfortable feeling caused by the rusty appearance can be reduced or even ignored.

Acknowledgements

The author would like to acknowledge the sponsorship of the Ministry of Education, through Capes Foundation and the EcoLanes Project, FP6.

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Accelerated Load Testing of Rigid Road Structures under Simulated Traffic (I)

PhD Student Marius-Teodor MUSCALU

Summary

The present paper presents the partial results of the study related to the deformations of the BcR rigid road structures and RCC roller compacted concrete, tested on the Accelerated Load Testing (ALT) facility from the Technical University of Iasi Road Station. These tests were carried out in the frame of the EcoLanes FP6 Project : “Economical and Sustainable Pavement Infrastructure for Surface Transport”.

The pavements are plain and steel fiber reinforced concrete SFRC, the reinforcement being realized with steel fibers recovered from used tires.

The transducers types are described and the results of the tests under a dynamic load of 57.5 kN, corresponding to the standard load specified by the Romanian norms are interpreted.

KEYWORDS: ALT, BcR rigid road structure, roller compacted concrete, steel fibers from used tires, steel fiber reinforced concrete, transducers.

1. INTRODUCTION

This paper presents the main aspect of the experimental study of the performance of various rigid pavements under accelerated traffic undertaken in the frame of EcoLanes Fp6 Project [1] seeking the development of economical and sustainable pavement infrastructure for surface transport.

This ongoing study is conducted on the circular Accelerated Load Testing facility of the Road Research Station at the Technical University "Gh Asachi" of Iasi, where various rigid pavement structures are subjected to a repeted ALT standard load of 115 kN.

The main objective of this project is the development of pavement infrastructure for surface transport using classic and roller compacting technology [2] in considerations with concrete mixes reinforced with steel tire-cord fibers recovered from post-consumed tires.

2. THE ROAD RESEARCH STATION

The ALT-LIRA facility from UT Iasi, as shown in figure 2.1, is the third generation [3] and its construction has been imposed by the adoption, in Romania, of the standard axle OS-115 for pavement structural design.

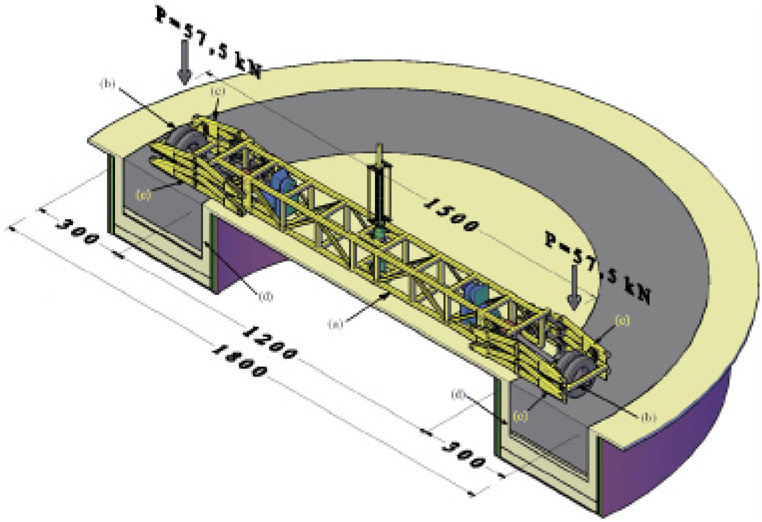

Fig. 2.1. The ALT-LIRA facility from Technical University „Gh. Asachi" Iasi: a) the main steel structure; b) the wheel assambly; c) supporting girders; d) the circular sink

3. EXPERIMENTAL PAVEMENT STRUCTURES

In relation with figure 3.1 and with the aim to develop the technology of rigid pavements constructed with concrete reinforced with steel fibers recovered from tire recycling, on the ALT circular track the following pavement structures have been constructed:

- unreinforced road concrete slab (BcR – road cement concrete), base course layer of ballast stabilized with cement and the subbase layer of ballast (sector 1);
- road concrete slab reinforced with recovered steel fibers (BcR+SRSF), the base course layer of ballast stabilized with cement and the subbase layer of ballast (sector 2, 3 and 4);
- unreinforced roller compacted concrete slab (RCC) , the base course layer of ballast stabilized with cement and the subbase layer of ballast (sector 7);

- roller compacted concrete slab reinforced with recovered steel fiber (RCC+SRSF), the base course of ballast stabilized with cement and the subbase layer of ballast (sector 5 and 6).

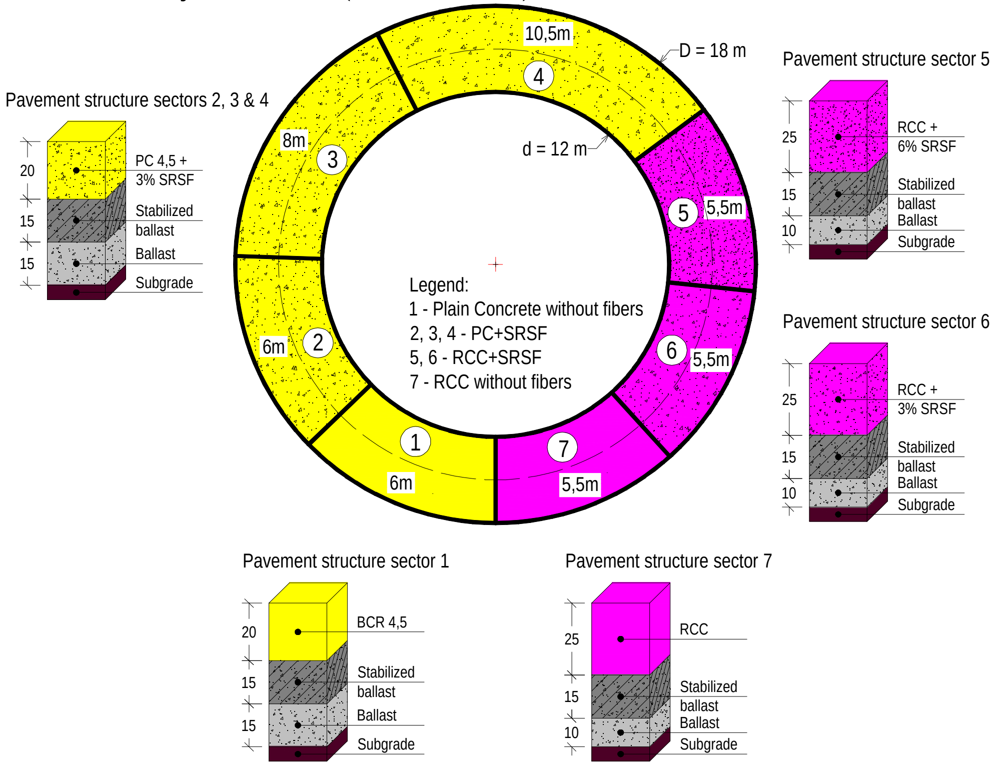

Fig. 3.1. The pavement structures constructed on the ALT-LIRA facility

The composition of the unreinforced and reinforced concrete used for the construction of various slabs and their mechanical characteristics are given in the Annex 1.

The composition of the unreinforced and reinforced RCC and their mechanical characteristics are given in the Annex 2.

4. THE TRANSDUCERS USED IN THE EXPERIMENTAL PAVEMENTS

In order to determine the concrete slab deformations and pressures at the level of subgrade layer, the experimental sectors have been equipped with 28 strain transducers type PAST 2-AC (PAvement Strain Transducers for Asphalt Concrete), and 3 transducers for measuring the pressure at the level of the subgrade layer type SOPT 68A (SOil Pressure Transducers).

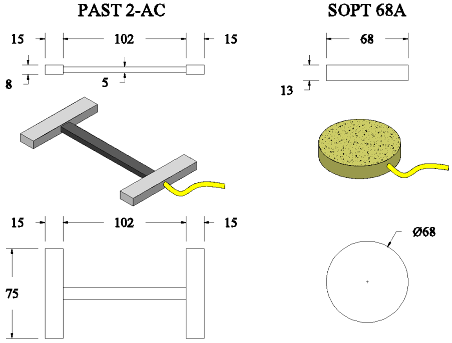

Fig. 4.1. The shape and dimensions (in mm) of the tensometric transducers

SOPT 68A transducers were placed on the level of the subgrade layer, on the center line of sectors 2, 3 and 4, before the execution of the ballast layer. The PAST 2-AC transducers were placed with their axis at a distance of 2.5 cm from the base of the concrete slabs, during construction. In Figure 4.2 are presented the positions of the transducers.

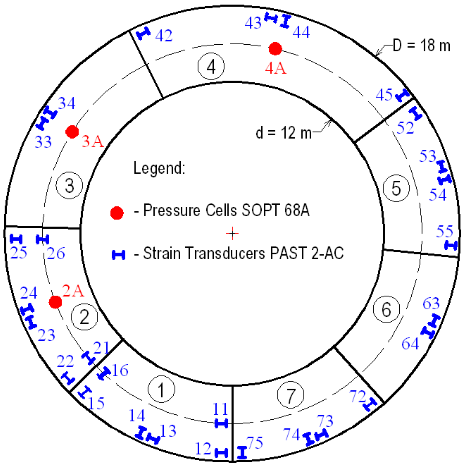

Fig. 4.2. Placement of the transducers on the experimental sectors

5. THE RESEARCH PROGRAM

Concerning the simulated traffic, it is envisaged to achieve at the end of the experiment 1.500.000 passes of the 57.5 kN load. The stage analyzed in this paper refers to 200.000 passes.

In accordance with figure 5.1 the circulation is performed on three different circular trajectories having rays of 7.20m; 7.50m (track axis) and 7.80m.

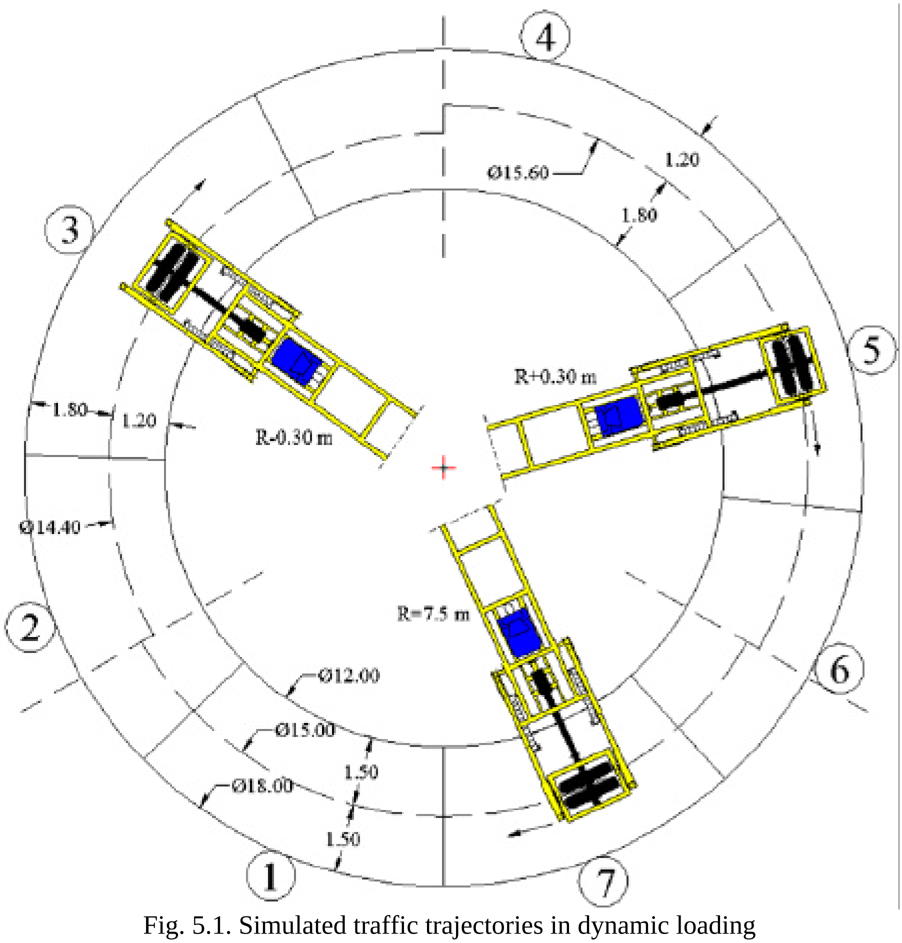

Fig. 5.1. Simulated traffic trajectories in dynamic loading

The pressure cells are monitoring the behavior of the BcR 4.5 concrete slabs depending on their length. The strain transducers are monitoring the behavior of the reinforced concrete slabs compared with those unreinforced, by recording strains in the most critical positions established in accordance with literature [4]:

- position one: the center of the slab is not considered as a critical one;

- position two: at the edge of the slabs; this position is representative for the reinforced/unreinforced concrete slabs in the hypothesis of uniform distribution of slabs on the support base realized with cement stabilized ballast;
- position 3: at the corners of the slab.
- position 4: at joints;

Strains are recorded in both longitudinal and transversal directions.

6. EXPERIMENTAL DATA

In the Annex 3, are presented the results measured by the transducers equipped on sector no. 2 in dynamic loading at the stage of zero passes of the wheel load.

In the Annex 4, a comparison between the measured deformations and pressures at different traffic stages under dynamic loading is presented.

Additional deflection tests with the Heavy Weight Deflectometer (HWD) are also envisaged to be undertaken during the second stage of the ALT experiment. The surface roughness and the technical condition of the slabs under ALT traffic will be also investigated.

7. CONCLUSIONS

- the accelerated load test is performing with the standard load of 115 kN, a load of 57.5 kN being applied on the double wheel;
- it is envisaged that the accelerated load test will continue until achieving 1.500.000 passages
- comparison between BCR and RCC unreinforced and reinforced with steel fibers recovered from waste tire recycling (SRSF) is seeking the confirmation of the usefulness of adopting RCC technology and determining the class to which technical solution is appropriate technical and economic;
- the deformations recorded with PAST transducers in the significant loading positions (2, 3 and 4) according to literatures [4] will be completed with the results of the Heavy Weight Deflectometer (HWD) in order to establish the elastic modulus and the edge behavior (rigidity and rate of transmission);
- the significant wearing of wheel tires during the first stage of the ALT experiment justifies the need of applying of an asphalt protection layer over the surface of the RCC slabs;

ANNEX 1: Composition of unreinforced and reinforced concrete used for the construction of various slabs and their mechanical characteristics

Tab. A1.1. Unreinforced BcR mix design.

Quantities per 1 m^3 of concrete		
Natural aggregates	%	kg
Natural river sand 0 - 4	40	748
Natural river sand 4 - 8	18	337
Chippings 8 - 16	22	411
Chippings 16 - 25	20	374
Total dry aggregates	100	1870
Cement I 42.5 R		350
Water		146
Additive Cementol Zeta T concentrate	0.67	2.33
Additive Cementol ETA S	0.20	0.73
Concrete density, kg/m^3		2369

Tab. A1.2. BcR mix design reinforced with 3% fiber.

Quantities per 1 m^3 of concrete		
Natural aggregates	%	kg
Natural river sand 0 - 4	45	807
Natural river sand 4 - 8	27	485
Chippings 8 - 16	28	502
Total dry aggregate	100	1794
SRSF fibers	3.0	69
Cement I 42.5 R		360
Water		158
Additive Cementol Zeta T concentrate	1.0	3.60
Additive Cementol ETA S	0.1	0.36
Concrete density, kg/m^3		2417

Tab. A1.3. Mechanical characteristics performed on laboratory samples.

Test	Age, days	Strengths, MPa		Allowable limits for BcR 4.5 according to NE 014-2002
		Sector 1	Sector 2 - 4	
Bending test on prisms, 150x150x600	7	4,0	5,1	-
	28	5,6	6,3	4,5
Compression test on cylinders,D=150 mm, H=300 mm	3	13,5	18,2	-
	7	18,4	25,5	
	28	24,0	28,6	
Compression test on cubes, 150x150x150	7	30,1	36,5	-
	28	45,6	51,5	44

ANNEX 2: Composition of the unreinforced and reinforced RCC and their mechanical characteristics are given in the Annex 2.

Tab. A2.1. RCC mix designs for the experimental sectors.

RCC without fibers			
No.	Material	%	Kg per 1 m^3 of compacted concrete
1	Natural river sand	25	505
2	Crushed river sand	30	606
3	Chippings 4-8	25	505
4	Chippings 8-16	20	404
5	Cement I 42.5R		300
6	Water	6.5	151
RCC with 3 % SRSF fibers			
No.	Material	%	Kg per 1 m^3 of compacted concrete
1	Natural river sand	25	512
2	Crushed river sand	30	615
3	Chippings 4-8	25	512
4	Chippings 8-16	20	410
5	Cement I 42.5R		300
6	SRSF fibers	3	70
7	Water	6.5	157
RCC cu 6 % fibre SRSF			
No.	Material	%	Kg per 1 m^3 of compacted concrete
1	Natural river sand	25	495
2	Crushed river sand	30	594
3	Chippings 4-8	25	495
4	Chippings 8-16	20	396
5	Cement I 42.5R		300
6	SRSF fibers	6	140
7	Water	7	170

Tab. A2.2. Mechanical characteristics performed on laboratory samples.

Test	Age, days	Strengths, MPa		
		RCC without fibers	RCC with 3% fibers	RCC with 6% fibers
Flexural test on prisms, 150x150x600	7	5.50	4.79	3.47
	28	7.00	5.90	5.44
Compression test on cylinders,D=150 mm, H=300m m	7	19.37	9.80	9.53
	28	21.44	13.16	13.11
Compression test on cubes, 150x150x150	7	41.30	43.47	23.65
	28	50.30	54.75	31.63
Compression test on prism ends	7	35.76	29.32	21.45
	28	40.79	39.58	30.46

ANNEX 3: Results measured by the transducers equipped on sector no. 2

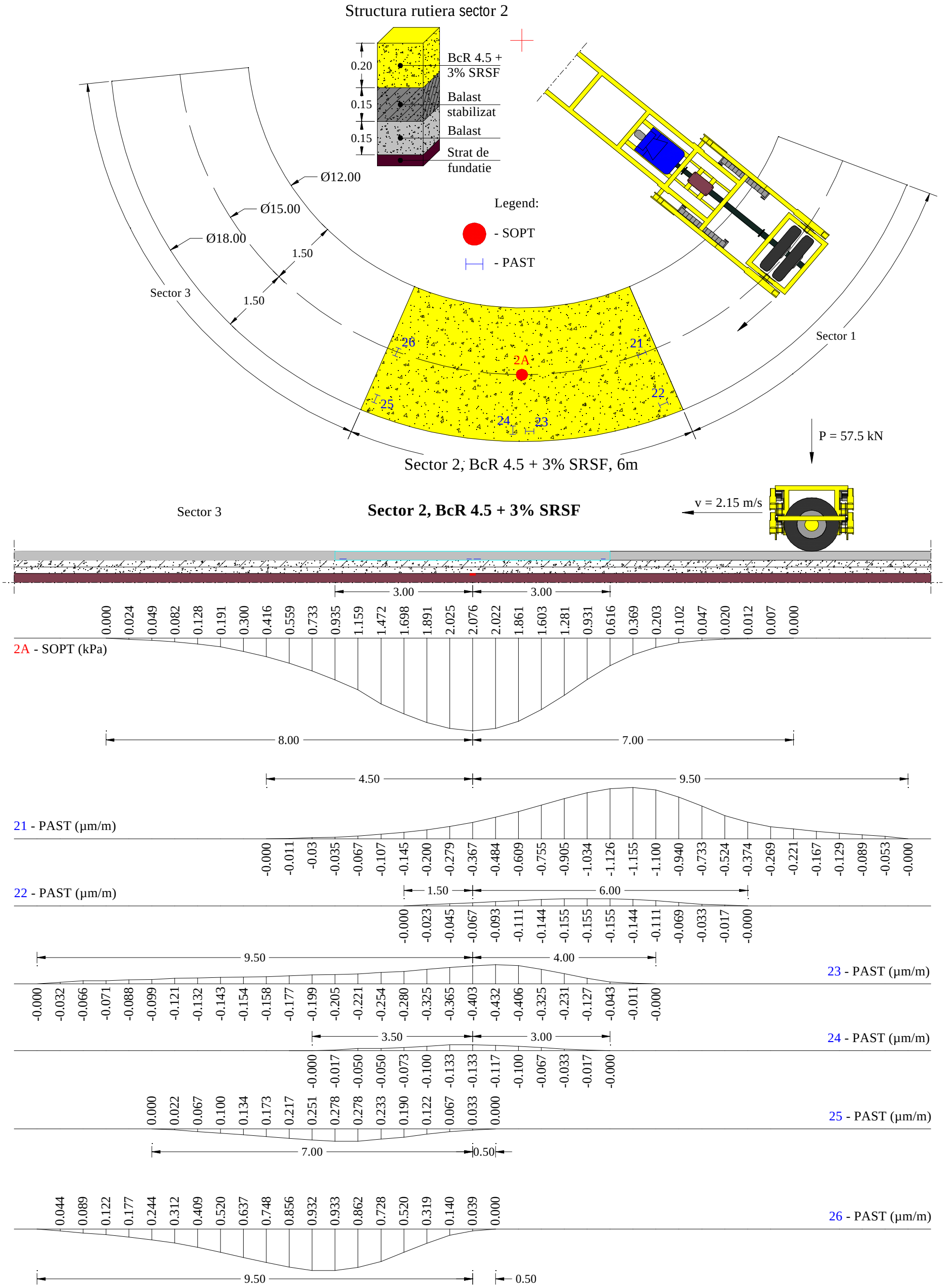

Fig. A3.1. Dynamic loading for sector no. 2, at the stage of zero passes of the wheel load

ANNEX 4: Comparison between the measured deformations and pressures at different traffic stages

Tab. A4.1. Evolution of the deformations and pressures at different traffic stages under dynamic loading

Sector no./ Concrete type	Passes	Longitudinal deformations, μm/m				Transversal deformations, μm/m				Pressures, kPa			
		Trans .no. *)	Radius **), m			Trans .no. *)	Radius **), m			Trans . no. *)	Radius **), m		
			7,20	7,50	7,80		7,20	7,50	7,80		7,20	7,50	7,80
	0		-0,764	-0,778	-0,683		0,231	0,167	0,100				
	100000	11	-	-0,900	-0,725	14	-	0,200	0,200				
	200000		-0,900	-1,000	-0,833		0,133	0,233	0,167				
	0		-0,505	-0,400	-0,400		1,416	1,754	2,567				
1/BcR	100000	12	-	-0,500	-0,433	15	-	3,150	3,890	-	-	-	-
	200000		-0,700	-0,800	-0,633		2,350	2,633	3,550				
	0		-0,510	-0,589	-0,644		-0,227	0,233	0,233				
	100000	13	-	-0,535	-0,533	16	-	0,300	0,300				
	200000		-0,533	-0,700	-0,700		1,000	0,600	0,267				
	0		-1,000	-1,155	-1,350		-0,133	-0,133	-0,133				
	100000	21	-	-0,775	-0,786	24	-	-0,267	-0,180	-	-	-	-
	200000		-1,025	-0,850	0		-0,167	0,100	0,200				
	0		-0,246	-0,155	-0,267		0,400	0,278	0,434		1,980	2,076	1,862
2/BcR	100000	22	-	-0,400	-0,300	25	-	0,467	0,325	2A	-	2,775	2,600
	200000		-0,267	-0,333	-0,433		0,250	0,367	0,400		2,790	3,338	3,140
	0		-0,559	-0,432	-0,489		1,011	0,933	0,906				
	100000	23	-	-0,167	-0,250	26	-	1,550	1,167	-	-	-	-
	200000		-0,100	-0,625	-0,425		3,100	2,800	1,900				
	0		-0,692	-0,655	-0,675		0	0	0		3,971	3,387	3,224
3/BcR	100000	33	-	-0,467	-0,550	34	-	-0,125	-0,180	3A	-	5,015	4,300
	200000		-0,433	-0,500	-0,500		0,133	0	0,133		5,145	5,425	4,725

Continuing of table A4.1.

4/BcR	0	42	-0,300	0	-0,251	44	0,594	0,659	0,633	4A	5,108	4,798	4,052
	100000		-	-0,367	-0,825		-	0,333	0,140		-	6,690	5,725
	200000		-0,733	-0,750	-0,800		0,467	0,200	0,300		6,650	6,875	5,833
	0	43	-0,533	-0,417	-0,854	45	0,690	0,972	1,136	-	-	-	-
	100000		-	-0,250	-0,375		-	1,475	1,650				
	200000		-0,367	-0,500	-0,550		1,067	1,535	1,833				
5/RCC	0	52	-2,003	-2,054	-2,373	54	1,889	1,984	2,028	-	-	-	-
	100000		-	-1,333	-1,257		-	1,300	1,475				
	200000		-1,467	-1,267	-1,167		0,650	1,233	1,375				
	0	53	0,600	0,500	0,500	55	-0,816	-0,533	-0,477				
	100000		-	0,967	1,425		-	2,333	3,000				
	200000		1,467	1,175	1,280		1,700	2,325	3,150				
6/RCC	0	63	-	0,527	-	64	-	0,533	-	-	-	-	-
	100000		-	1,333	1,950		-	0,567	0,633				
	200000		0,933	1,325	2,100		0,367	0,675	0,767				
7/RCC	0	72	-0,293	-0,280	-0,390	74	0	0	0	-	-	-	-
	100000		-	-0,667 0,433	-0,750 0,950		-	0,233	0,300				
	200000		-0,600	-0,800	-1,133		0,200	0,333	0,400				
	0	73	0	0,184	0	75	1,228	1,453	1,967				
	100000		-	0,575	0		-	2,133	2,467				
	200000		0,500	0,500	0,350		1,475	2,000	2,300				

*) – see Fig. 4.2.
**) – see Fig. 5.1.

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Practices of road authorities an important factor in increasing road safety

Sachelarie A.[1], Birsanescu P. [2], Zarojanu Gh. [3]

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Summary

This paper presents an evaluation of traffic safety on the roads of romania and the importance of actions taken by institutions with activities in the field of traffic safety

Keywords: road safety, road authorities, black points

1. INTRODUCTION

It is a reality that developing countries and countries with economies in transition have limited capabilities to address road safety, and stresses the importance of international cooperation in the field of road safety collaborative process involving policy-makers, representatives of nongovernmental organizations, and academics from around the world.
World Health Organization statistics shows that a crash is ranked 9th among the causes of mortality worldwide and place among the causes of generating violent death. Annually lose their lives on roads the world over 1.2 million people, some 50 million suffer serious injuries, often with sequelae throughout life.

2. ASSESSMENT OF TRAFFIC SAFETY

As well, the explosive development of the fleet on the one hand, and permanent increase in the number of holders of driving licenses have transformed the driving into a social phenomenon, road accident, as part of this phenomenon, a became a major factor of risk. Statistics show unfortunately only worrying dimensions of the phenomenon, but not traumas and enormous costs that are behind them.
If we want to quantify, the concept of road safety, it may be reduced to reporting the number of road events according to the categories of vehicles involved, the period during which the event occurred, the victims of the crime scene, etc.. Such a

quantitative analysis of traffic events may be introduced mathematical formula in the form of indices which take into account the severity of the event (damage to property, light or serious injury, death) and the indices that quantifies the number of accidents which have caused deaths, serious injuries, light or material damage.
The road transport system could be considered [4] as a triangle of three key components which are user, vehicle and road (Fig.1). Each of these factors can contribute individually to traffic accidents and not only one single element is responsible for an accident, but combinations or interactions between them.

- Between vehicle and road as physical and geometrical factors, which are well defined in several technical guide lines. They are the tools of road engineers [2];
- Between user and vehicle as man-machine-interface. The ergonomical demands of the drivers are subjects of the car industry [1];
- Between user and road as Human Factors [6].

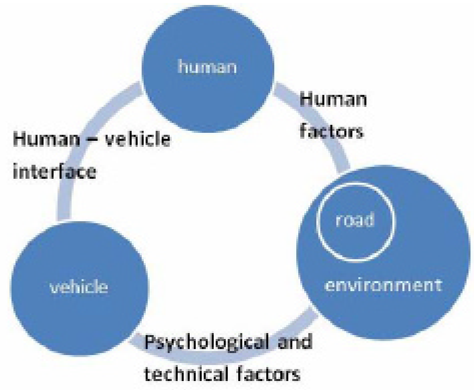

Fig.1. Traffic safety triangle

The human - vehicle - road is a complex system whose ties manifest themselves in both directions interacting and allowing the state system. One of the most effective methods of reducing the number of maneuvers wrong, as long-term, consists of the changes to the road surroundings. Therefore, must be taken into account the entire design, maintenance and extension frame surrounding the road, having regard to human factor and the ability to deal with traffic.
A more realistic approach to the conventional components of the basic road safety, referring to an essential element - the code road or traffic legislation [4], (Fig.2). One can say that on these four components are always involved in various programs initiated by government agencies or non-governmental, but progress in the sphere of reducing the number of traffic accidents registered locally are reduced. A conclusion which unfortunately is a certainty, is that drivers are almost the only ones who can contribute to reducing road events with serious consequences.

Through a imprudent driving, excessive speed unsuitability to traffic conditions, failure to obtain priority, driving under the influence of alcohol or overruns imprudente, are just some of the causes generating situations with a high degree of risk on the vast majority of participants road traffic.

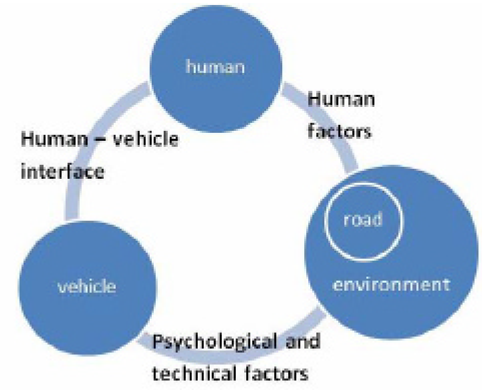

Fig.2 Traffic safety tetrahedron

Another aspect that can not be ignored when discussing the concept of road safety, more specifically countries, with some hesitation in the transition to European integration, is the security on such criminal acts, mostly directed to the area carriers of commercial goods, and the upper range of automotive luxury (the destruction or theft of a vehicle or goods, or killing the drivers or passengers).

An important factor in increasing the safety of traffic on public roads incumbent authorities to act in this regard by every means:

- Directorate General of Romanian Police (DGPR)
- Road Police Department (SPR);
- Romanian Road Authority (ARR)
- National Company for Motorways and National Roads in Romania (CNADNR)
- The National Union of Road Hauliers from Romania (UNTR)
- Insurance Companies;
- Technical Universities etc.

As it is reasonable [1], first of all need to identify the main risk factors. SPR, CNADNR, UNTR, after processing the data in the field, have drawn in a first phase, map of black points on the roads in Romania (Fig. 3). ("Black Point" - 10 accidents with at least 10 victims in 5 consecutive years - fatal or serious - on a portion of 1 km sector road).

After processing the data in the field, have written in the first same data [10], so they could show the most dangerous roads in Romania reported the no. road event and the main causes of producing serious accidents [5].
Table 1 and and figure 4 shows the situation of serious accidents on the main road in Romania, reported in the period 2007-2008.

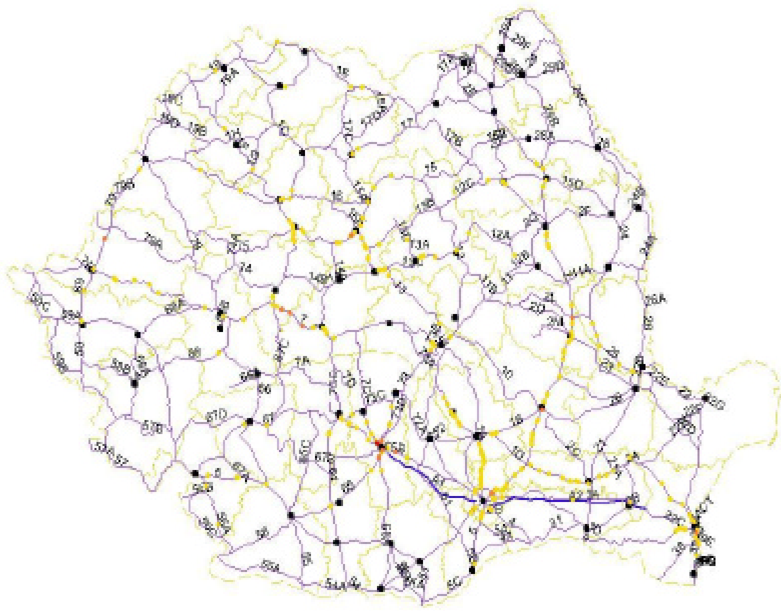

Fig. 3 The map of black points on the roads in Romania

Table .1 The situation of serious accidents on the main road in Romania, reported in the period 2007-2008

Road	Accidents			Dead			Serious injuries		
	2007	2008	Dif	2007	2008	Dif	2007	2008	Dif
DN1	**408**	**437**	**29**	**114**	**110**	**-4**	**215**	**209**	**-6**
DN2	**322**	**368**	**46**	**81**	**87**	**6**	**101**	**160**	**59**
DN7	291	309	18	76	90	14	151	178	27
DN6	282	252	-30	63	69	6	92	87	-5
DN15	110	138	28	37	31	-6	48	63	15
Others	4372	5799	1427	1274	1425	183	4199	5780	1580
Total	5785	7303	1518	1524	1707	199	4806	6476	1670

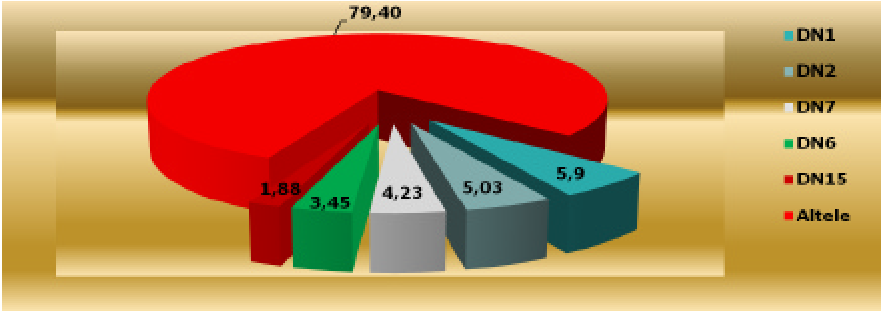

Fig. 4. The situation of serious accidents on the main road in Romania, reported in the period 2007-2008

Table 2 and and figure 5 shows the main causes of road accidents on the roads in Romania, reported at the same time.

Table 2. The main causes of road accidents on the roads in Romania, reported in the period 2007-2008

Main cause	Accidents			Dead			Serious injuries		
	2007	2008	Dif	2007	2008	Dif	2007	2008	Dif
Irregular crossing pedestrians	1207	1324	117	383	315	-68	833	1021	188
Unsuitability speed to road conditions	709	1094	388	279	395	116	653	1051	398
Imprudent driving	551	606	55	198	208	10	464	512	48
Failure priority vehicles	497	625	128	106	95	-11	467	623	156
Failore priority pedestrians	480	572	92	74	86	12	418	501	83
Others	2341	3082	738	855	995	140	1971	2768	797
Total	5785	7303	1518	1895	2094	199	4806	6476	1670

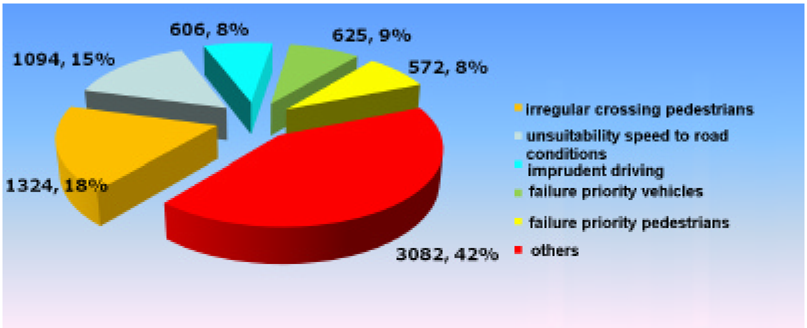

Fig. 5. The main causes of road accidents on the roads in Romania, reported in the period 2007-2008

Through "STOP ROAD ACCIDENTS! LIVE HAS PRIORITY! ", initiated by the Directorate General of Romanian Police, have developed a variety of preventive activities, which the police have informed people about the importance of this ongoing program and tried to realize the general public of the importance of the life before speed (Fig. 6)

Fig. 6 "STOP ROAD ACCIDENTS! LIVE ARE PRIORITY! "

SPR, CNADNR and Local Administrations, through "STOP ROAD ACCIDENTS! WILD has dynamic road accident victims and results were passed to the "*Protocol of cooperation in increasing the circulation flow and road safety on public roads*", to reduce the victimization of people by road traffic accidents. Under this protocol, the during 2008, was implemented a system of monitoring traffic on national roads (Fig 7), creating a special compartment in the SPR to supervise and monitor the traffic, finding and applying criminal sanctions contraventional or, where appropriate, with the preparation and submission documents by drivers who have committed irregularities in the movement.

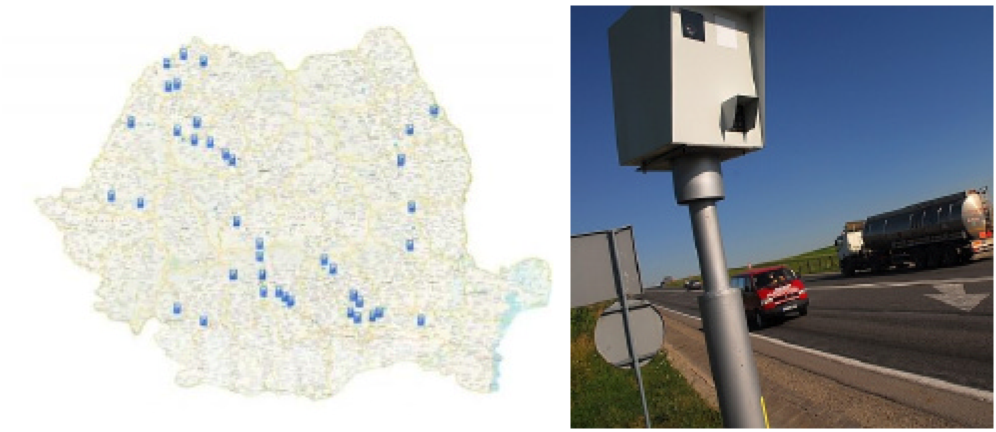

Fig. 7 The map of places where they are assembled, and one of the models of equipment used.

From the total traffic level, almost 20% represents the driving by night, in hard conditions, when the variable and cyclic intensity level of the headlamp illumination of the opposite side drivers decrease the seeing capacity of the driver.

More than 33% of traffic accidents are done in those conditions. Knowing that one of the common excuses drivers involved is "blindness", Laboratory Department of Automotive and IC Engines in collaboration with the SPR measurements were conducted to determine the traffic of vehicles with defects in the light. The importance of our study [7] consist in the determination of the automotives in motion with a high level of headlamp low beam illumination that can produced a short time blind driving situation, as is well known that the automotive lighting system has an essential contribution to the increase of traffic safety and stress reduction under reduced visibility conditions. At the end of our study we evaluate, 300 vehicles running on a district way during the night time. The observations were done in three periods, during 2005 and 2007, each time were observed 100 vehicles. The results are synthesize in table 3 and figure 8.

Table 3. The vehicles with a high level of low beam headlamps illumination

Obs.	Vehicles with a high level of low beam head lamps illumination	Old vehicles	Without adaptive front lighting system	Trucks or agricultural vehicles	Technical inspection out of date
2005	35	28	23	6	2
2006	26	26	21	6	-
2007	24	22	19	4	-

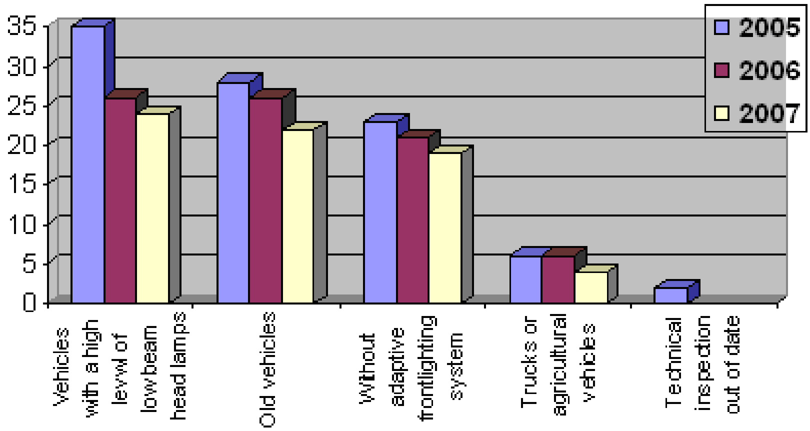

Fig. 8 The structure of vehicles with a high level of low beam headlamps illumination

The experience accumulated in the area of road accidents and events, allows us to affirm that about 80 percent of serious road accidents and about 94 percents of the events that produce only material damages, have the human factor as cause [8]. In these serious accidents and events, causes as driving errors have a significant percentage, followed by the accidents due to poor technical status of vehicles. The man behind the wheel, as in life, has a clearly distinguishable behavior. In this context, this refers to the driver capacity in adapting and reacting to different situations which may place him in conflict with other people or may affect his health. The emotional reactions, when facing an accident danger that may end in own injuries or injuries to other traffic participants, are approximately the same when considering the reaction time and the avoiding maneuvers, even taking into account all the differentiations imposed by the social and cultural level. This conclusion is valid as long as human behavior is not influenced by some perturbing factors (e.g., driving under the influence – alcohol or drugs –, neurological or nervous illnesses). Following a partnership with three insurance companies, has conducted a study on "predisposition of drivers in road accidents.

By analyzing a large number of accidents produced by the same persons, it may be observed that the main factors would be:

- the nervous functional plasticity, whose negative effects on new situation adaptability determines inadequate reactions;
- lack of movement coordination;
- actual intelligence deficiencies;
- higher reaction speed compared to sensing speed that determines an anticipation of movements, without having all pertinent information at hand.

Therefore, it was considered that the term “predisposition” does not reflect the reality, and, as a consequence, it was replaced with the more pertinent term for this phenomenon, “accidents susceptibility”, reflecting a temporal behavior of the person.

A clear distinction has to be made between temporary and long-term predisposition to accidents. For example, the temporary one that may be caused by alcohol consumption may pass in a few hours, while the long-term one is permanent regardless. The idea that the accident susceptibility is influenced by some sensorial deficiencies of the individual was opposed by behavioral analyses of drivers with visual deficiencies that did not caused accidents, due to the increased prudence in driving style to compensate for the physical deficiency.

Unfortunately, our statistics do not reflect the simple vehicle collisions (fender-bender), with only material damages. However, from data acquired between 2000 and 2002 from several insurance companies (928 cases), it may be concluded that in about of 12.6% (117 cases) of simple accidents with material damages only involve drivers that have prior records of accidents (at least one prior collision in the recent past). About 61.53% of these drivers are up to 35 years old, and 19.65 % are of average age (Fig 9). Only 8% of them are women, the large majority being men.

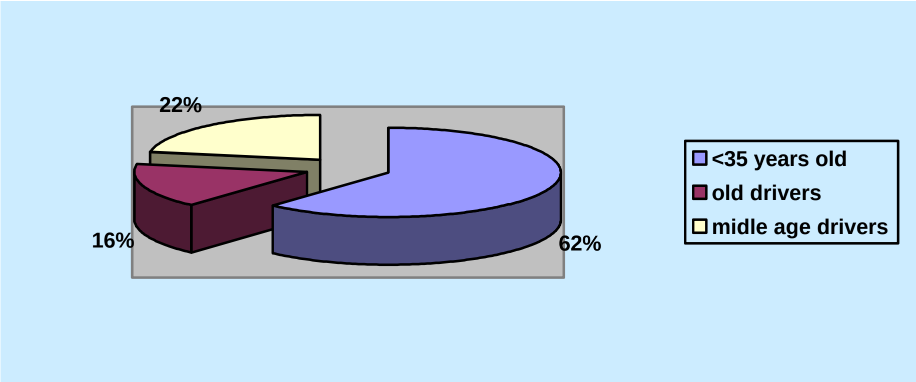

Fig. 9 The structure of “dangerous drivers”

Our study consists in processing statistical data from only three insurance companies and is not relevant whether those „dangerous drivers” were also involved in litigations with other insurance companies or had other insignificant problems (for our analysis) with the traffic police dept., such as penalties and fines.

Another partnership (ongoing) ended with the administration of a local joint located on E85, the range which was a record number of road events involving victims, the partnership is to finalize the measures that will lead to improved traffic. As a first step to measure the speed of movement in the vicinity of pedestrian crossings, found that over 75% of vehicles transiting the area exceeded the maximum speed allowed on the settlements.

3. CONCLUSIONS

Dynamics analysis of accidents reported, that, based on the damage road indiscipline among pedestrians, but also imprudent, failure to limit the speed, etc.. among drivers, the number of road accidents remains at high. On may concluded that to reduce the number of traffic accidents required to be taken a number of measures that have the effect of traffic discipline participants:

- To increase road safety and accident prevention and combating of the traffic is necessary as the exact identification of the causes of the conflict generating road;
- Popularization of concrete involved in road events, highlighting the causes and the consequences of accidents;
- Improved co-operation with road managers, with specialists in the systematic and forecast road traffic.

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Highway and Bridge Engineering 2008 in images

Photo 1. Opening session – Prof. Neculai Taranu – Dean of School of Constructions Iasi

Photo 2. Opening session - Prof. Constantin Ionescu

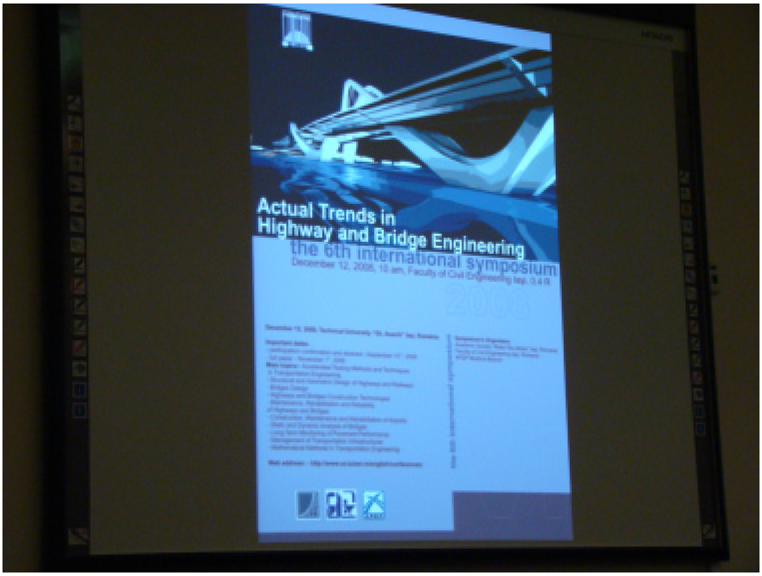

Photo 3. The begin

Photo 4. First presentation

Photo 5. PhD Blejeru Cristian - Presentation

Photo 6. Prof. Constantin Jantea – Points of view

Photo 7. Prof. Andrei Radu – Presentation

Photo 8. Image from the audience

Photo 9. Image from the audience

Photo 10. Image from the audience

Photo 11. Image from the audience

Photo 13. Image from the audience

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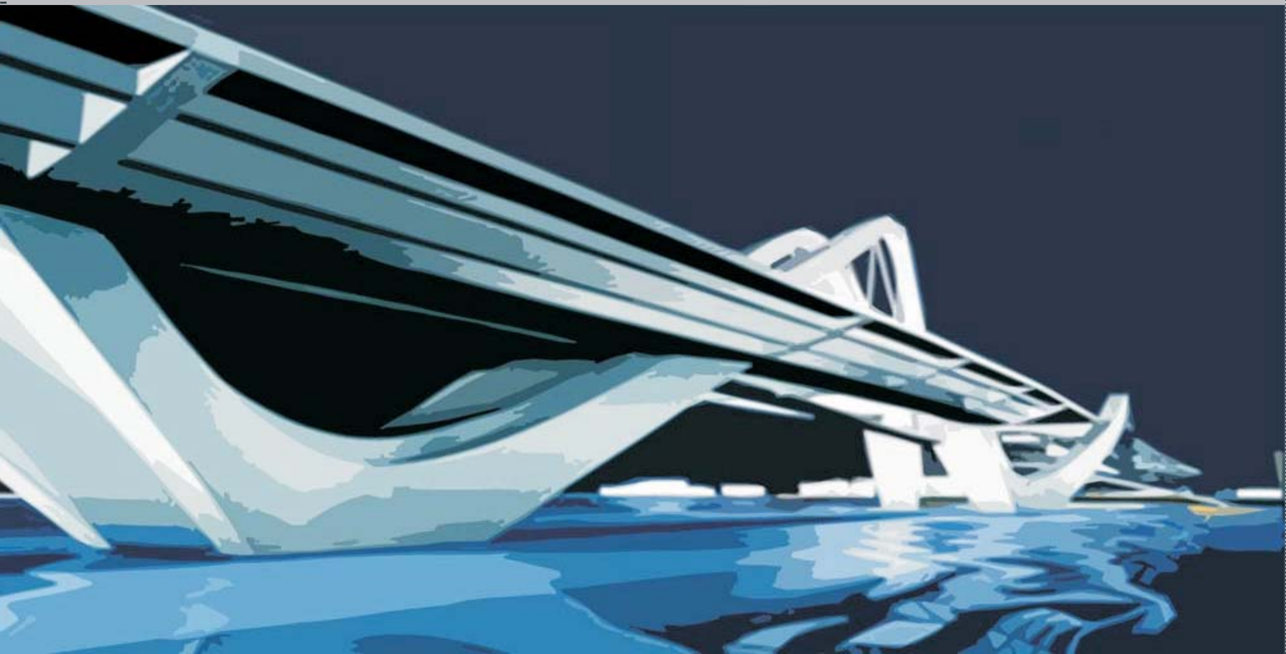

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