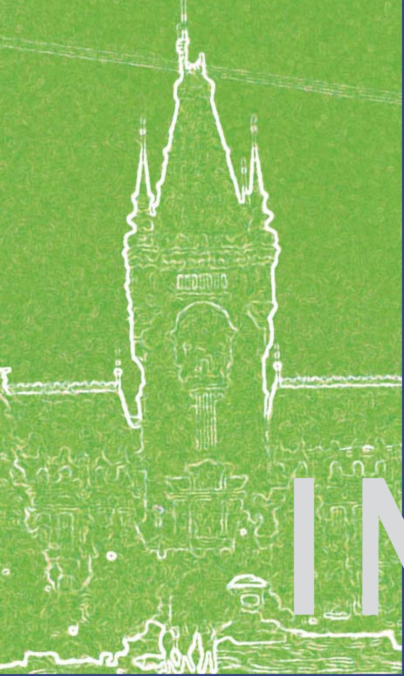

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Letter from Editor

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This third annual issue of ***Transportation Infrastructure Engineering****, hosted by the electronic edition of the* ***Intersection Journal,*** *is dedicated mainly to the relative young engineers and researchers, from our country and abroad , also to doctor and master candidates confronted with specific research and new ideas, trying to emphasize their significant endeavor and contributions they are bringing in the challenging field of transportation research.*

This issue is opening with a succinct presentation of significant results of the research works, undertaken in the frame of ***the Department of Civil Engineering, University of Minho from Portugal.*** *This introductory paper entitled:* ***The Road Network Rehabilitation for 21-est Century – a Global Vision on Innovation in Road Rehabilitation" ,*** *drafted by the group of researchers (Jorge Pais, Elisabete Freitas, Hugo Silva and Joel Oliveira), conducted by* ***professor Paulo Pereira****, intends and succeeds to be* ***"an open approach to road network rehabilitation and its impact on the pavement life"*** *in the context of the new challenges for the near future. In their, endeavor to offer the society, better roads based on conceiving and implementing a sustainable construction and rehabilitation of road pavements, the Portuguese researchers are also emphasizing the role of creative thinking and innovation at the service of society, in the field of road engineering.*

*With their paper: "****Reliability and Durability of Concrete and Pre-stressed Concrete Bridges , Decision Making Process and Risks****", our Czech colleagues* ***Jiri Pokorny, Vladimir Dolezel, Josef Stryk and Karel Pospisil*** *, from The Transport Research Institute of Czech Republic, are describing the most frequently occurred failure causes within the realization, operation and reconstruction of the bridge structures, recommending some interesting non-destructive tests, in order to be used in bridge diagnostics.*

Vasilica BEICA, PhD Eng., *from Romanian Centre for Road Engineering Studies and Informatics –CESTRIN, presents the results obtained , during her doctoral study, on the evaluation of various bituminous binders used in rehabilitation and modernization works, involving the SHRP/Superpave equipments and specifications in conjunction with the classical method in the paper :* ***"A Performance Grade Polymer –Modified Bitumen, According to SHRP Specifications".***

Another distinguished specialist ***Mihai Stasco, PhD Eng.,*** *presents in his paper the research work undertaken during his doctoral study, on the "****Electrical Simulation of the Rheological Behavior of the Asphalt Mixes".***

The next six papers are dealing with the research issues, undertaken, at Universidade do Minho, (http://www.eng.uminho.pt) in Portugal, under the

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guidance of a group of Portuguese Tutors, coordinated by ***Professor Paulo PEREIRA*** *and* ***Professor Paulo B. LOURENÇO,*** *by each of our MSc students involved in the Leonardo da Vinci, mobility project RO/2004/PL 93209/S :* *"****Students Vocational Training – Premise for an Integrated European Transport Infrastructure".***

This project, initiated by our University, has been conceived as a trans-national mobility action, having as main objective, the placement, during the begining of the year 2006, of six Romanian students, from Technical University "Gh. Asachi" Iasi, to the Department of Civil Engineering/School of Engineering/ University of Minho, located in Guimarães, Portugal. The tasks envisaged for the Romanian students was to perform there, a combined academic research and vocational training stage of five months, directly related with the subject of their diploma/ master works. The papers are short extracts from their dissertation works (see the web-site: *http://www.ce.tuiasi.ro/~ccf/events.html*).

*In their paper: "****Modelling of a Flexible Road Pavement in Portugal****", the authors* ***Andrei Gabriel Ionescu*** *and* ***Elisabete Fraga Freitas*** *present the works undertaken in the frame of "Leonardo da Vinci" Mobility Project: RO/2004/PL93209/S, at Universidade do Minho - Center for Civil Engineering, Highways Laboratory, under the coordination of Paulo Pereira, PhD, PE, Professor of Civil Engineering, tutored by Elisabete Freitas and Jorge Pais. This article presents the solving of one of the issues in this project: the modelling of an existent pavement with its pavement condition and bearing capacity. Further, under the same relevant tutoring,* ***Irinel-Diana Vrancianu*** *and* ***Elisabete Fraga Freitas*** *, present the solving of one of the significant issues of the project:"* ***The Division of the Road Tested with FWD into Homogenous Sectors, Checking the Homogeneity and the Statistical Relevance of the Division – According to COST 336 Action Final Report****".*

Costel Cristian Botezatu, Joel Oliveira *and* ***Hugo Silva,*** *in their paper****: "The Main Stages of Road Infrastructure Concessions in Portugal****", refer to a portion of the new highway build in the Porto area. This road is part of the Concession Scut do Grande Porto that congregates a group of freeways and groups road associates in the area of Grande Porto, integrated in the National Road Plan, his work being undertaken in the frame of the same Leonardo mobility project RO/2004/PL93209/S realized at University of Minho from Portugal.*

Vlad Apreutesei, Daniel V. Oliveira *and* ***Paulo B. Lourenço****, in their paper:****"Repair and Strengthening Techniques for Masonry Arch Bridges"*** *describe some of the techniques used in the process of strengthening damaged masonry arch bridges. The problems are very complex because existing bridges differ in structural materials, in construction age, in types and in condition rating. The most frequent bridge building materials have been stone, wood, reinforced concrete and steel. Existing stone bridges are centuries-old and many of them are*

Letter from the Editor

historical buildings. Any repair has to take into account not only defects and damages identified, but also the main features of the bridge, the intervention costs and operation difficulties.

***Cristina - Emanuela Lanivschi**, **Graça Vasconcelos** and **Paulo Lourenço** present their research results on "**Numerical Analysis of Historical Construction**", with practical application to the restauration of the Porto Cathedral in Portugal.*

*Finally, closing this third issue, two enthusiastic graduate students from our University, **Anca-Aura Gavrilitza** and **Cristina-Lucia Lucache**, are presenting their work study, concerning the behaviour and performance of some experimental road sector realized on the existing road network in their comprehensive paper: **"Performance of the Experimental Road Pavement Sectors Realized with Asphalt Mixtures Sstabilized with Various Fibers and Improved Bitumen on National Road NR 17 Vatra Dornei- Suceava".***

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The road network rehabilitation for the 21st Century. A global vision on innovation in road rehabilitation

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Summary

This article intends to be an approach to road network rehabilitation in the context of the new challenges for the near future. Firstly, the current methodology used in the maintenance and rehabilitation and its impact on the pavement life cycle costs is presented. Secondly, the role of innovation at the service of society in the field of road engineering is presented in order to assure a high level of the ride quality, as well as a sustainable construction and rehabilitation of road pavements.

1. INTRODUCTION

Road networks require to be preserved in terms of infrastructures (pavements, bridges, road marking, road signs and safety equipment), using rational maintenance and rehabilitation strategies, which basically would consist in applying the "3 R's strategy": "*the Right treatment, on the Right road, at the Right time*" (FP[2], 2001). The development of this strategy is supported by updated and accurate road information related to the road infrastructure performance, allowing the analysis of data which characterize the condition of the road network, as well as the development of maintenance and rehabilitation strategies, considering given quality standards, or, as an alternative, taking into account available financial resources.

In the road network the pavement constitutes the most important infrastructure as it is submitted to important factors, such as traffic and climate, which have major consequences on the pavement performance. For this reason authorities dedicate significant investments in the construction, maintenance and rehabilitation of road networks which, consequently, become a fundamental domain of research.

Road pavements are designed to support traffic and climatic actions over a certain life time (20 to 40 years, with the objective of offering safe and comfortable ride conditions).

Having into account technical, economic and environmental perspectives for the structural and functional quality standards, after a pavement is constructed for a certain life time, every operation in the infrastructure should be minimised with the

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objective of reducing costs related to its quality maintenance by considering the different stakeholders: i) road administration; ii) users; iii) environment.

Thus, in this context, roads should be seen as infrastructures that should accurately allow a comfortable and safe riding. Moreover they should cause little impact on the environment, what would contribute for the improvement of the quality of life of rural and urban areas.

However, road pavements frequently require premature or non-scheduled maintenance and sometimes reconstruction operations before they reach the end of their lifetime, for which they were designed, with significant costs for every stakeholder.

2. THE REHABILITATION OF THE ROAD NETWORK AND ITS IMPACT ON THE PAVEMENT LIFE CYCLE COSTS

The management of a road should be considered throughout its entire life cycle (20 years average). Hereafter, two rehabilitation strategies are analysed together with the consequences for the road quality and global costs.

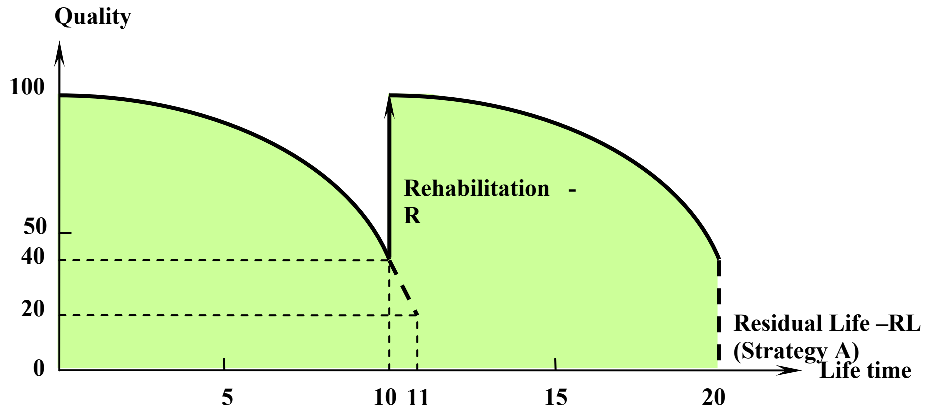

Figure 1 – Pavements life cycle

In general, two alternative strategies could be contemplated: i) maximising the global quality (area of Figure 1), considering available financial resources; ii) minimising the costs for the road administration, for the user rand and for society in general (energy costs), considering given quality standards, being the latter the proactive strategy.

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Figure 1 represents the strategy which comprises a periodic rehabilitation in addition to the current maintenance operations (strategy A). The highlighted area represents the global quality obtained for the overall life cycle. With this strategy the pavement will still show a residual life of RLA at the end of its life cycle.

Photograph 1 – Cracked pavement (alligator cracking): pre-failure condition

Meanwhile, the current practice typically comprises reactive rehabilitation strategies, which act essentially at the structural level, without considering functional components, as it is undertaken when the pavement has already lost its "structural capacity". This type of strategy has a highly negative impact on the life cycle costs of pavements for society, particularly for users.

Considering the pavement shown in Photograph 1, which presents alligator cracking pattern at an initial condition of failure, without apparently showing a severe reduction of its bearing capacity, a complete diagnosis would allow defining future rehabilitation in order to avoid the total loss of its structural capacity.

Alternatively, if nothing is done, this pavement will evolve until complete failure, as it already shows alligator cracking and disaggregation (Photograph 2). This will contribute for an increase of permanent deformation as a result of the ingress of water through the cracks into the granular layers and subgrade. Under these conditions this pavement will require an urgent and costly reconstruction.

In order to support the study of the impact resulting from the adoption of a proactive rehabilitation strategy, an urban road of high traffic capacity with an Average Annual Daily Traffic (AADT) of 40000 to 60000 will be considered, as shown in Photograph 3. In Photograph 4 the high deteriorated condition of the pavement, which had already been repaired, is shown.

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Photograph 2 – Pavement in failure condition

This case study would have been chosen for a strategy of construction-rehabilitation privileging the construction of a pavement of high structural capacity for a long life time, minimising the rehabilitation operations.

Photograph 3 – Urban road of high traffic capacity

Facing a premature degradation of the pavement quality, a deep rehabilitation is needed to diagnose and to eliminate the existing problems. In these situations, by acting at the surface course level only the functional quality will be improved, disguising structural problems temporarily. In addition to direct administration

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costs, these operations will have significant costs for the users. Medium and short term rehabilitation will affect the global costs.

The adoption of a reactive-type strategy, acting only when the residual life of the pavement is very low (Strategy B; Figure 2), will have a significant impact on the costs during the pavement lifetime. Hereafter, these costs are evaluated and compared with those resulting from the strategy defined in Figure 1 (Strategy A).

Photograph 4 – Deteriorated pavement of a high traffic capacity urban road

Rehabilitation strategy B (Figure 2) could be considered as a set of three types of operations: two light rehabilitations, R1 and R3, and a deep rehabilitation, R2, leading to a residual value of RLB. In this strategy B, a quick evolution of the deterioration was observed if compared with strategy A.

Taking into consideration these two rehabilitation strategies, the consequences regarding the following parameters were determined and analyzed:

i) global quality of the pavement, related to all costs and, particularly, vehicle operating costs (VOC);

ii) user costs, represented by the cost concerning the modification of travel time, and by the costs of fuel, as these are significantly relevant;

iii) environmental costs, partially related to the green house effect and to energy consumption.

This case study has a length of 2000 metres, with a cross-section of 2x3 lanes, and a total width of 21 metres, what means that the total area to be rehabilitated is 42000 square metres.

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The value for AADT would be 40000 vehicles: two users per vehicle, totalising 80000 people a day; fuel consumption: 10 litres per hour and vehicle; hourly cost: 25.00€.

Relating the costs of the rehabilitation operations, that of reference, R, (Figure 1) will have a unit cost of 3.00€ per square metre. Rehabilitations R1, R2 e R3, will have a unit cost of 1.50€, 4.00€ e 1.50€, respectively (Figure 2).

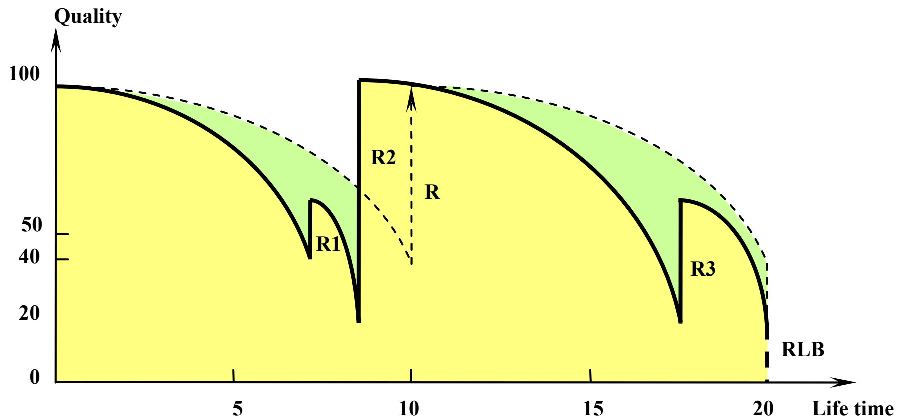

Figure 2 – The consequence of two rehabilitation strategies on the pavement life cycle

Concerning the duration of each rehabilitation, and the corresponding time modification, the following values are considered: i) R1 and R3 with the duration of 3 days and an increase in travel time of 0.2 hours per day of operation; ii) R2, as well as R, will have a duration of 5 days, resulting in an increase in travel time of 0.3 hours per day.

The results obtained with these calculations for strategies A and B are presented in Table 1.

Table 1 – Consequences of two rehabilitation alternatives strategies

Strategy	Rehabilitation costs	Increase on the travel time	Travel time costs	Fuel costs
A	126000€	120000 h	3000000€	780000€
B	294000€ (+133%)	216000 h (+80%)	5400000€ (+80%)	1404000€ (+80%)

Considering the added value of strategy A compared with B, it is possible to compute its impact in terms of the road network length that would be possible to rehabilitate, at a preventive rehabilitation level (R1 e R3 rehabilitation type, with an unit cost of 1.50€ per square meter).

The total added value of strategy A is 168000€ in terms of direct costs for rehabilitation and 3024000€ regarding the direct costs for users (travel time and fuel consumption). Considering a preventive rehabilitation of a road with two lanes, with a width of 7 meters, that amount of added value will allow the rehabilitation of 16 km and 288 km, respectively.

In addition to the significant increase of costs for strategy B, the pollution costs resulting from more than 96000 hours of fuel consumption of a single vehicle should be emphasised as well as the negative impact on the economy derived from an increase of 80% on the imported energy, the vehicle operating costs and the safety and comfort costs.

3. A GLOBAL VISION OF THE CONTRIBUTION OF ROAD REHABILITATION FOR THE QUALITY OF LIFE

In the context of a holistic approach to the quality of life, roads need to be analysed throughout their entire life time, including the following direct and indirect costs: i) construction and rehabilitation costs; ii) user costs, including those resulting from traffic delays due to maintenance and rehabilitation works; iii) environmental costs; iv) societal costs in general.

To accomplish this holistic objective of the road network, a new vision would assume the following predictions (FEHRL, 2004): i) roads will constitute an infrastructure which will promote technical and social developments to be base of a better quality of life; ii) construction and rehabilitation of road infrastructures will be sustainable if they include technical, economic, social and environmental dimensions; to support this objective it is fundamental to *establish the bridge:* "*design-construction-maintenance*"; iii) the road network will be constituted by a set of "*intelligent infrastructures*", in which "*5 star vehicles are driven along 5 star roads".*

In an urban environment, every operation should integrate the existing legislative requirements, namely regarding noise, searching for continuous solutions in terms of service for users. To reach to a solution a global analysis comprising structural and functional components in order to improve the quality offered to users and non users and to minimise the environmental impact needs to be made.

Thus, a new attitude is required: a proactive vision of the road network management in the framework of modern Road Network Management Systems. This activity requires design monitoring, followed by the construction phase, the monitoring of the performance of the road network constructed until the new cycle of maintenance and rehabilitation, according to predefined quality standards.

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This innovative approach should be global, taking into consideration the factors evaluated and the road stakeholders, trying to innovate in order to "*make better with less*".

With this perspective in mind, promoting the structural sustainability of the road infrastructure for a long term is essential. Therefore, it would be possible to concentrate only in the innovation at the level of the surface course, searching for added value for users and non users, particularly in the field of safety, bearing in mind the interaction "*driver-vehicle- road*". At the same time this approach would integrate the concept of "pavements eco-efficacy" for any rehabilitation.

The objective of maintaining a certain level of quality standards of a road network throughout its entire lifetime requires an appropriate road network management in order to analyse the following: i) factors to be evaluated; ii) direct and indirect stakeholders; iii) added value for each management policy.

Any road network rehabilitation study should comprise the following factors: i) road characteristics: road category; traffic volume; ii) marginal occupation; iii) global potential impact of rehabilitation operations; iv) stakeholders (road administrations, users and non users). The practice of integrating the different categories of users in this approach is not usual. However, as they are the main supporters of the overall costs involving any road operation (especially through the taxes they pay) their integration is essential for this process.

For every category of stakeholder the added value of this approach is evident: i) the improvement of the service offered by the road to users in terms of travel time - determinant factor for classifying the quality of the road network; ii) the reduction of the cost for the road administration - less operations and high efficacy of those ones; iii) the reduction of the user and non users costs (the population in general receiving the impact of road and every activity related to its construction and rehabilitation (air and sound pollution); iv) the reduction of environmental costs (air, sound and water pollution); v) the reduction of energy costs.

4. THE ROAD ENGINEERING INNOVATION AT THE SOCIETY'S SERVICE

The starting point for a prospective analysis of innovation in Road Engineering would be the statement "technically everything is possible" (FEHRL, 2004). For example, the development of "intelligent vehicles" will continue forcing the evolution of the road category to the level of "intelligent road": "in the future the road will command the vehicle".

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For this approach the research to be undertaken would integrate multidisciplinary competences in the fields of Road Engineering (materials, behaviour, design, performance monitoring and road management), Communication and Information Technology (CIT) and Electronic Engineering. Thus, roads will appeal to innovative materials as well as to intelligent monitoring methods. That would make possible to establish the functional relation “road> manager> user”.

4.1. The Innovation at the Travel Quality Level

The road has to serve society in general. Thus, any operation at the level of its main infrastructures (pavement, marking and safety equipment) requires sustainable knowledge about how this infrastructure behaves at the interaction of “driver-vehicle-road” (Figure 3).

At the level of riding safety, the measures to reduce accidents should constitute a multidisciplinary effort shared by all the intervenients. Among those measures there would be a wide set of activities such as the development and management of the infrastructure, the promotion of safer vehicles, the enforcement of law, the preparation of health services, and the transportation planning.

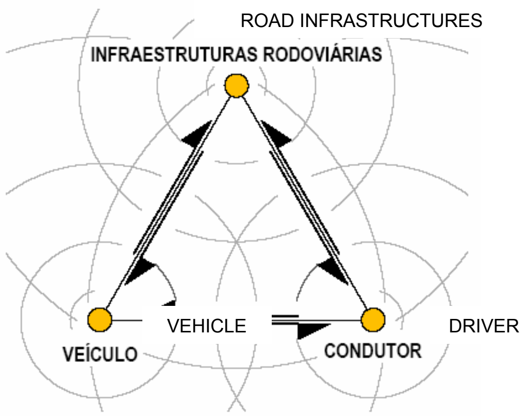

Figure 3 – The interaction “driver-vehicle-road” (adapted from Martins & Nabais, 2006)

The research developed at an international level reveals that drivers are held the main responsible for accidents, followed by the road environment, and, last but not least, the condition of vehicles (Austroads, 2003; Rothengatter & Huguenin, 2004).

At the European Union level, the Project SUNflower+6 (*Comparative Study on the Development of Road Safety in Nine European Countries*), co-financed by the European Commission and leadership by the institute SWOV of the Netherlands, carried out a comparative study of road safety (Figure 4) comprising the following

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nine countries: Sweden, United Kingdom, Netherlands, Hungary, Check Republic, Slovakia, Portugal, Spain and Greece; and in the autonomic region of Catalonia (Macedo et al., 2006).

In these countries, there are still serious problems regarding road safety, such as: i) drunk driving; ii) insufficient child protection inside the vehicles; iii) accidents involving young drivers (mainly at weekends and at night); iv) driving in excess of speed in all types of road; v) a high rate of pedestrian accidents, mainly for children and elderly people; vi) the severity of accidents involving two-wheel motorcycles.

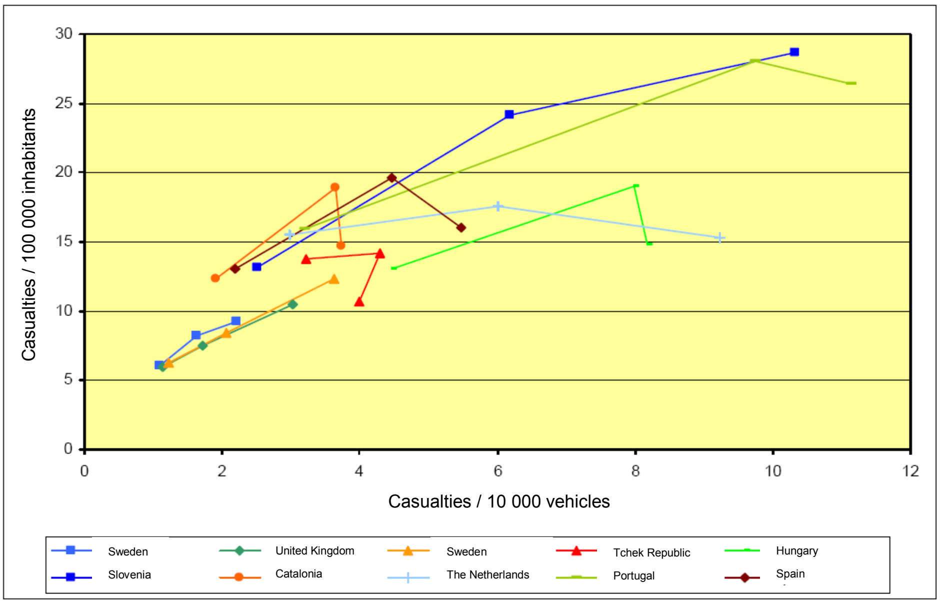

Figure 4 – Road casualties in the European Union (Macedo et al., 2006)

The recommendations arisen from the experience of this project for the European Commission are: i) the reinforcement and the improvement of the efficiency of the supervision; ii) the improvement of means and procedures for the acquisition and treatment of data related to accidents and the drivers' behaviour; iii) the application of measures and operations in road safety as well as the monitoring of its results; iv) the education, training and recycling of the different road users and other intervenients in the various areas related to road safety.

Meanwhile, in addition to the intervention of the driver in every field of road safety, the quantification of the effects of any corrective measure in the road environment is of great importance, as it allows defining the criteria for supporting any decision regarding the most effective measures.

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The existing factors which direct or indirectly provoke road accidents are: i) requests to which drivers are subjected inside the vehicle (use of cell phones and audio equipments as well as the rolling noise; ii) the outside interferences for drivers (publicity boards; iii) meteorological conditions (rainy weather is directly related to road accidents; it may also affect the physical and psychological conditions of drivers); insufficient light; iv) the road alignment and its geometric parameters such as width, slope gradients, shoulders, etc. which determine the driver's behaviour regarding speed; v) the pavement condition - roughness and surface friction of pavements are determinant factors of riding and comfort and safety.

The way drivers detect these factors is essential to develop solutions which allow warning drivers of any changes on the road environment, in particular on the pavement surface.

Riding quality and comfort are intimately bound. Noise also becomes an important factor, if we consider that affects users and non users (neighbourhoods near the "road influence area"). In this context, innovative projects in road surface courses, responsible for ensuring the functional quality of pavements, have been developed what has had a direct impact on the users and on the environment.

In Europe, the current methods used to reduce road noise include noise barriers, traffic control (speed limitation) and the alteration of vertical and horizontal alignment as well as the definition of protected zones.

An innovative method to obtain noise reduction is through "silent pavements" (Camomilla & Luminari, 2004), as the present state-of-the art on vehicle technology does not foresee a significant reduction of the vehicle motor noise and its exhaustion system. The porous surface courses, and more recently, the "*twinlayer*" surface course, have been used in several countries as a noise reduction measure (Hofman & Kooij, 2003).

Noise reduction can also be obtained through the utilization of thin layers such as the "*Poroelastic Surfaces*" (Fujiwara et al., 2005), conceived to control the texture and voids, introducing new materials, such as rubber, and new pavement concepts as the "*Ecotechnic Pavement*" and the "*Euphonic Pavement*" (Camomilla & Luminari, 2004).

Thus, any operation in the urban road environment needs to optimise the choice of the surface course with the aim of reducing the "*road noise influence area*". Figure 5 shows the evolution of the "noise map" after the application of an innovative surface course with a bituminous mixture incorporating tyre recycled rubber modified bitumen.

In addition to this type of impact on the pavement rehabilitation, some others such as the surface drained water quality or the air quality have to be evaluated.

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Figure 5 – Comparative noise map of the rehabilitation consequences (Nobre, 2006)

4.2. Sustainable Construction and Rehabilitation of Road Pavements

Over the last few decades, there has been an increasing concern about the limitation of operations at the level of the surface course in the field of pavement rehabilitation (Nunn, 1997). The surface course may be object of a set of different rehabilitation alternatives, which assure the different functions of the pavement, being a privileged field for fundamental and applied research.

Innovative surface courses need to be resistant, durable and to offer surface conditions capable of guaranteeing ride safety and comfort for the users and economy. At a global level, they need to ensure environmental quality over their life time. In addition to these capabilities, this type of layers would also integrate intelligent referencing and monitoring systems, based on CIT, interacting directly with the management system and with the road user by assuring permanent high quality riding conditions.

However, the implementation of an innovative programme for surface courses also demands changes at the structural level of a pavement, under the leading direction of "*Sustainable Construction and Maintenance of Road Pavements*".

In this context, the structure of pavements has to be durable, by integrating innovative materials, and "environmentally friendly", by incorporating industrial waste materials, including pavement planings. In this way, all over the life time, monitoring costs *in situ and in the laboratory* will be reduced and the accuracy of structural behaviour models will be improved.

Thus, it is necessary to promote the structural sustainability of pavements, aiming at "*perpetual pavements*", which only require superficial periodic rehabilitation. At same time, it should also be assumed that a pavement should be managed as a structure in a "*close cycle*": materials used in the construction phase should be reused in posterior phases, minimizing any rejection of materials or the use of new materials over its life time.

4.2.1. Pavement Recycling

For a new approach in the field of Maintenance and Rehabilitation of pavements, recycling will play a fundamental role in the life cycle of pavements. As a consequence, research on this field is being focused on maximizing the incorporation of used materials in every rehabilitation action, as well as in new constructions.

Hereafter, two case studies related to the use of recycled materials are presented (Pereira & Picado-Santos, 2006): i) hot mix recycling; ii) cold recycling *in situ*, with bituminous emulsion.

The first case presents the following characteristics:

- The pavement structure is composed of a surface course of 6 cm of bituminous mixture (0/16), a bituminous macadam layer (0/25), 23 cm thick and a granular sub-base (0/50) with a thickness of 20 cm (Figure 6);
- The traffic for the new life cycle of the pavement is 40×10^6 ESALs (80 kN);
- The pavement has been in service for 7 years;
- The degradation shown by the pavement is top-down cracking reaching 10 cm depth; the granular layers present a good condition, being acceptable to consider a subgrade modulus of 60 MPa;
- The road to be rehabilitated is 10 km long, with four lanes. Total area to be rehabilitated: 160000 m^2.

The second case presents the following characteristics:

- The pavement structure is composed of a surface course (bituminous mixture, 0/16), 5 cm thick, a bituminous macadam base (0/25), 7 cm thick, one granular base (0/40), 20 cm thick and a granular sub-base (0/50), 20 cm thick (Figure 7);
- Traffic calculated for the initial life cycle was of 2×10^6 ESALs (80 kN);
- The pavement has been in service for 12 years;
- Every layer of the pavement is in severe conditions. Complete rehabilitation is required.
- From the analysis of the “in situ” characteristics of the pavement a modulus of 60 MPa was assumed for the subgrade;
- The road to be rehabilitated is 10 km long, with two lanes (total width: 8 metres), Total area to be rehabilitated: 80000 m^2.

In order to evaluate the benefits of the reusing the existing material (recycling) for each case study, two rehabilitation alternatives (milling/traditional overlay or recycling) were adopted. Thus, in the first case the solution consists of milling the

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upper 10 cm (with the transference of the milled material into a deposit), and applying new binder and surface courses. For the second case, a traditional overlay would be applied, including a stress absorbing membrane interlayer (SAMI) to retard crack propagation.

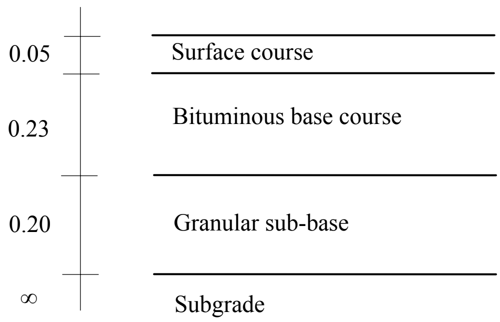

Figure 6 – Pavement structure for the first case study

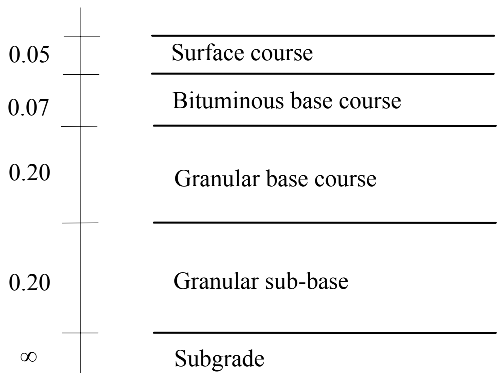

Figure 7 – Pavement structure for the second case study

Concerning the adoption of recycling techniques, milled material will be incorporated in the production (hot mix recycling) of a new bituminous mixture to be applied in the same pavement. In this way, it is possible to reuse 40% of milled material. In the second case the existing pavement will be recycled “in situ”, in a depth of 15 cm, with the addition of a bituminous emulsion with 3% of residual bitumen).

Pavement design for each alternative presented was undertaken using the programme BISAR (Shell, 1998). The results obtained for the thickness of new layers are presented in Table 2. Associated costs for each layer, and for the different alternatives, are presented in Table 3 and Figure 8.

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Table 2 – Thickness (cm) of the new layers for each alternative

Layer	Case study 1		Case study 2	
	Milling/Overlay	Recycling	Overlay	Recycling
Surface course	5.0	2.0	5.0	5.0
Binder or Base course	11.0	14.0	10.0	15.0

Table 3 – Associated costs for each rehabilitation alternative(in €)

Layer	Case study 1		Case study 2	
	Milling/Overlay	Recycling	Overlay	Recycling
Surface course	800000	352000	400000	400000
Binder or Base course	1232000	1097600	560000	240000
SAMI	--	--	80000	--
Total	2032000*	1449600	1040000	640000
Difference	582400 (28.7%)		400000 (38.5%)	

*The cost associated to dumping the milled material in a landfill is not considered (€20/m^3×16000m^3 = 320000€)

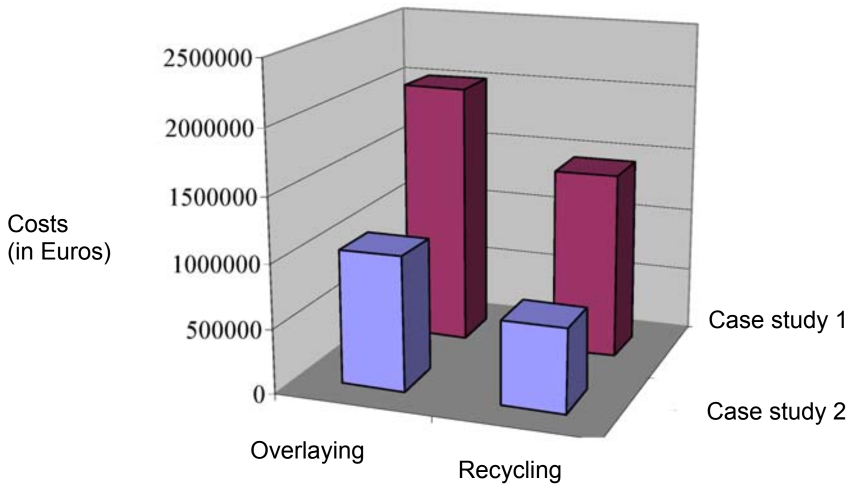

Figure 8 – Costs associated to each rehabilitation alternative

4.2.2. New paving materials

The search of pavement materials with a more accurate behaviour has to include the research of new high performance materials, assuming a minimisation of the maintenance and rehabilitation operations, having as a result the reduction of direct

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costs both for the administration and for users. In addition, the performance of these materials should also allow a reduction of the negative impact on the environment.

The search of high performance material requires the study of bituminous mixtures at the level of microscopic structures.

A micromechanical analysis of bituminous mixtures has been undertaken by several researchers (Buttlar & You, 2001; Silva, 2006). They take advantage of mechanical tests and microstructural models of discrete or finite elements to identify the main causes (at a micro level) of an inadequate behaviour (at a macro level).

According to Buttlar & You (2001), the microstructural modelling of bituminous mixtures is able to simulate the internal structure of aggregates and mastic, predicting accurately their behaviour.

For the study of the bituminous mixtures cracking, the use of a model with *lattice elements* is very efficient due to the importance that the bond between the materials involved (aggregate-aggregate, aggregate-mastic and mastic-mastic) has for the simulation of this phenomenon. In a simplified way, using a lattice model involves the approach to the continuous medium, where each element represents an intact link that could be broken during loading, forming a discontinuity (micro crack). Lattice models can be used to simulate a heterogeneous material, as it is the case of bituminous mixtures.

Recently, Silva (2006) developed a heterogeneous micro structural model made of lattice elements, used to study the evolution of a crack in the mastic and in the bituminous mixtures. The main results obtained with monotonic tensile tests were the tensile resistance, the failure strain and the tangent modulus, in different test configurations. Figure 9 shows the results obtained in the tensile tests carried out for the study of cracking in the mastic (a) and in the bituminous mixtures (b). These results were of fundamental importance for the development of predictive models of the mastic and bituminous mixtures.

Mastic and bituminous mixtures properties were utilised with lattice elements in the microstructural model in order to evaluate the influence of the micromechanical behaviour on the bituminous mixtures cracking. This model simulated the monotonic tensile tests carried out in the laboratory on bituminous mixtures. The aggregate and mastic distribution in the model was supported by digitised images of bituminous mixtures samples.

The factors that allowed the analysis of the quality of the bituminous mixtures predicted behaviour, using the heterogeneous models made of lattice elements, were: i) strength variation with the increase of the deformation, i.e. the non-linear response of bituminous mixtures; ii) cracking pattern observed in the samples.

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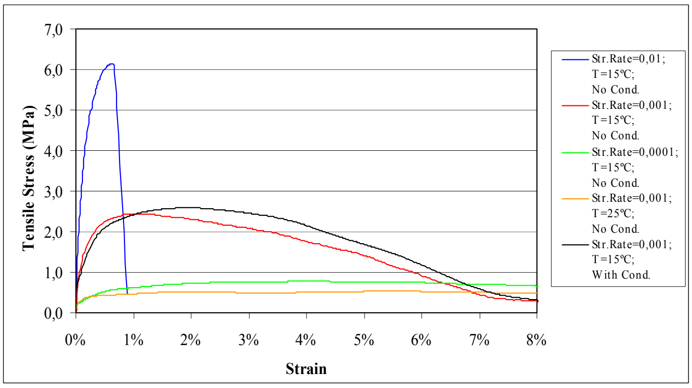
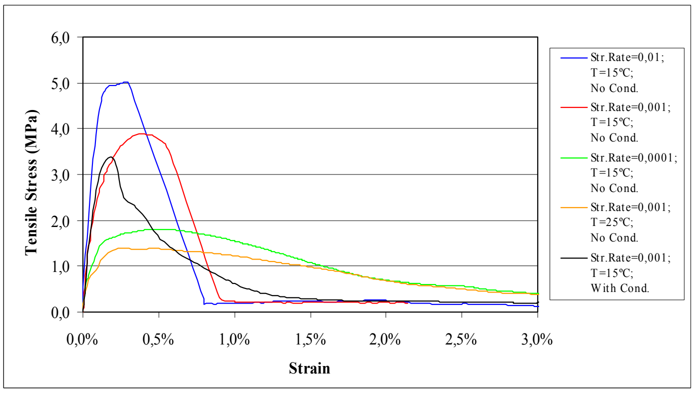


(a)



(b)

Figure 9 – Variation of the tensile stress of mastic (a) and of bituminous mixtures (b) as function of the applied strain

The behaviour obtained in tensile mode in predictive models was compared with that observed in laboratory. A good relationship between the numerical prediction and the experimental results was observed (Figure 10).

The comparative analysis of numerical and experimental results also allows evaluating whether the cracking pattern of models are similar to those observed in laboratory, as shown in Figure 11.

The visual analysis of cracking pattern allows concluding that, in general, cracking patterns observed in laboratory were well predicted by the lattice elements model.

The analysis of this numerical model allows researchers to observe that predicted cracking occurred always through the mastic, generally close to the aggregate surface, what proves the production of cracking in the bond aggregate-mastic due to internal cohesion problems of mastic.

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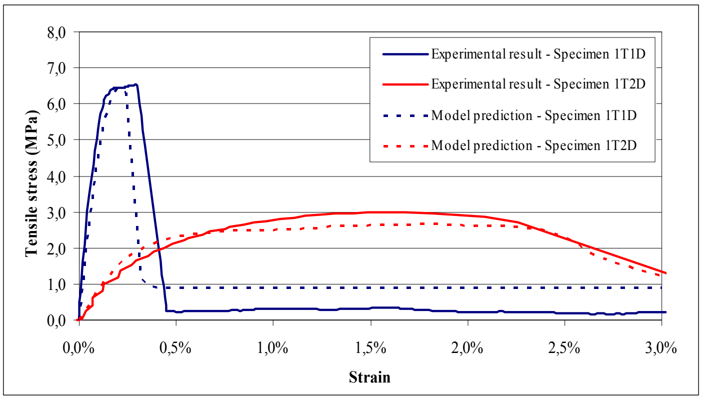

Figure 10 – Bituminous mixtures behaviour observed in laboratory and predicted by the simulation model

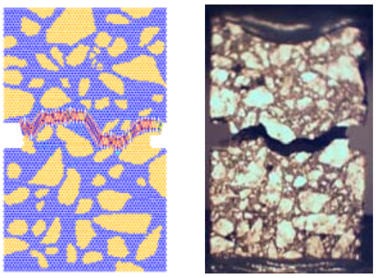

Figure 11 – Cracking pattern predicted in the numerical model observed in laboratory

The introduction of new materials in the constitution of pavement layers and the evolution of heavy vehicle loads configuration lead to the need of updating the current design methods as well as the specifications related to the behaviour of materials that should be demanded from innovative pavement structures.

With the objective of improving the performance of flexible pavements throughout their life cycle, the research of new materials has increased significantly, what means a reduction of future maintenance costs. A common alternative adopted in the field of road engineering is adding polymeric materials to bitumen aiming to improve its properties, mainly those related to thermal susceptibility and fatigue life (Neto, 2003).

Another concern associated to the improvement of the bituminous mixtures is the environment. Experiences with rehabilitated pavements in different countries have demonstrated the excellent structural and functional performance of bituminous mixtures with asphalt rubber modified bitumen.

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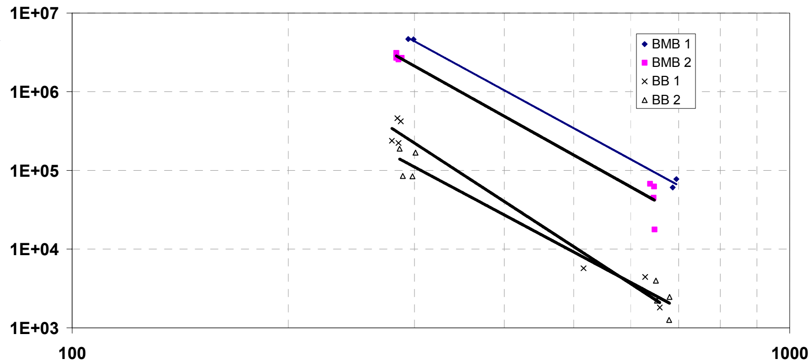

Figure 12 – Laboratorial performance of asphalt rubber and conventional bituminous mixtures

In the context of improving the behaviour of bituminous mixtures and reducing the environmental pollution, bituminous binders modified with rubber from used tires, known as asphalt-rubber, emerge. In general, with this type of new binder, significant improvements are observed: in the fatigue life as well as a reduction in the maintenance costs, an increase of skid resistance, a reduction of the cracking propagation phenomenon and a reduction of the noise level.

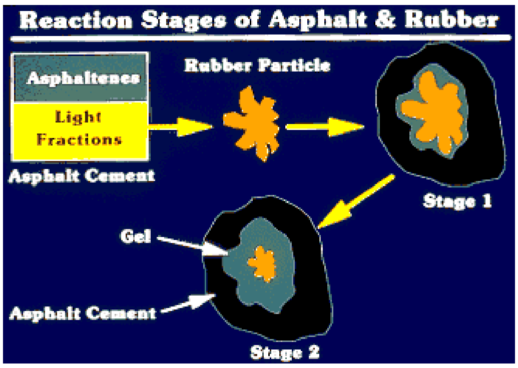

Figure 13 – Hypothetical model of the interaction between rubber particles and the conventional bitumen (Holleran & Reed, 2000)

Using fatigue tests in the four-point bending configuration, a difference in the performance between conventional mixtures and mixtures with asphalt-rubber was observed by Sousa et al. (2000), as shown in Figure 12.

The justification for the different performance of those two mixtures is supported by Holleran & Reed (2000). According to these authors, the asphaltenes and light fractions (maltenes, resins) of conventional bitumen interact with rubber particles.

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A reaction is claimed to occur, in which the asphalt and the rubber particle interact to form a gel coated particle (Figure 13). This mechanism allows fixing lighter fractions of bitumen, protecting them from climatic agents and avoiding their evaporation.

The application of numerical modelling to the design of pavement overlays allows to prove that mixtures with asphalt-rubber require a less thick overlay than those using conventional bitumen (Figure 14).

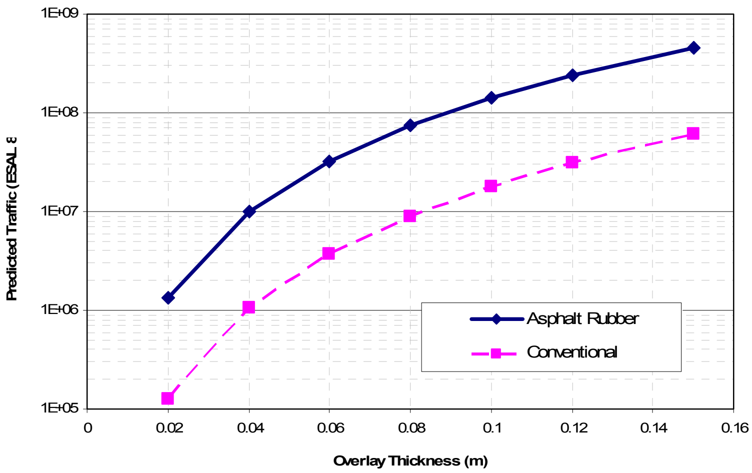

Figure 14 – Influence of bituminous mixture type in the overlay life (Sousa et al., 2005)

5. CONCLUSIONS

In the near future, the rehabilitation of the road network will assume a determinant importance and will lead to the adoption of a proactive vision in the field of road management. This global vision of rehabilitation will integrate all the components of the road network, as well as stakeholders.

In this context, innovation will assume an irreplaceable role by supporting road administrations, including road safety, to integrate the interaction "driver-vehicle-road" and the functional quality for users, non users and the environment. This approach will only be possible with the contribution of a sustainable strategy of construction and rehabilitation which will become part of a strategic cooperation among universities, road administrations and road related companies.

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Reliability and durability of concrete and pre-stressed concrete bridges, decision making processes and risks

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Summary

The paper deals with the reliability and durability of concrete and pre-stressed concrete bridges and with the decision-making processes and risks what to do with the bridge in a certain stage of life time. It describes the most frequently occurred failure causes within the realization, operation and reconstruction of the bridge structures. At the end there are mentioned some NDT methods, which are used in bridge diagnostics.

KEYWORDS: concrete bridges, reliability, durability, failures, safety, decision-making, NDT methods.

1. INTRODUCTION

Problems of the durability of concrete bridges and pursuit of the prolongation of their service life or optionally change in their usage is very topical matter today in particular from the following reasons:

- on considerable part of existing concrete bridges, evidences of ageing and damage start more and more to occur in the greater range that indicate oncoming exhaustion of load carrying capacity.

- demands on usability and resistance of bridges are growing rapidly today (the intensity of the traffic load is growing, axle loads of vehicles are increasing, by the damaging emission, the effects of environment on bridges are growing worse).

- construction of new objects is demanding on consumption of still rear raw materials and energies. Disposal of removed bridge structures is connected with a lot of technical and ecological problems.

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2. DURABILITY OF BRIDGES AND ITS SECURING (FACTORS INFLUENCING BRIDGE DURABILITY)

Durability of bridges is indivisible part of their reliability. The bridge has required durability, if it serves reliability conditions from the viewpoint of decisive limit states during the supposed life time T_∞ . These limit states are set down by actual codes and regulations. It can be possible only in that case if the initial bridge condition, given by the quality of project and realization, is on needed level and effects of load and environment will not during the time grow worse insomuch as the reliability system (structure, load and environment) will stop to work good due time $t < T_\infty$.

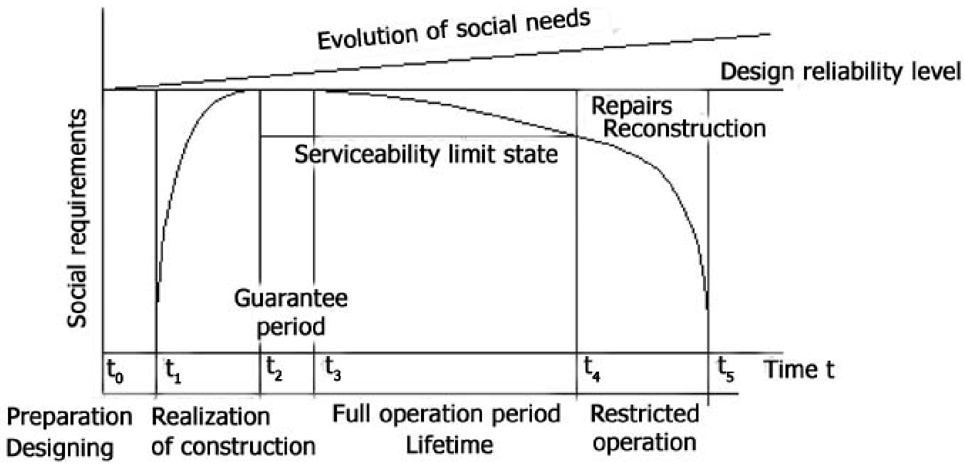

Fig. 1 Reliability and life time of bridge structure

On the basis of cause analysis of the bridge failures that occur the most frequently, it was found out the following percentage distribution of failure causes in the realization of bridge structure: according to the literature [1].

20% of failure causes are included already in the design of bridge structure (geological survey, project preparation and works connected with the preparation of realization).

50% of failure causes come directly under the process of bridge construction (material quality, wrong detail, lack of technological discipline).

15% of failure causes are possible to predicate directly to the operation.

15% of failure causes are possible to predicate to the other effects.

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If we analyse failure causes further we find the important fact that from all of the failure causes it is possible to predicate 80% of them to the human factor failure and only 20% are caused in the consequence of the bridge load.

From above mentioned it follows that construction of bridge object require high demands on qualification and quality of all the workers, namely during all stages of construction. On the Fig. 1 can be seen time intervals in life time of bridge structure.

The time t_2 to t_4 would meet our presumption about safe operational usage of bridge. Then decision-making process arrives what to do with bridge next.

3. RECONSTRUCTION OF BRIDGE

For user, it means to consider whether:

- modernisation of operation on the bridge is necessary and whether this arrangement will require intervention to carrying superstructure of bridge or

- reliability bridge condition (i. e. defects and failures) is of so much serious character that there is need to put a certain effort on its improvement within the framework of structure safety.

Basically it means two types of reconstruction that are joined together often in practise.

a) Reconstruction caused by change of operation is joined with the general repair of carrying bridge superstructure during that until now arisen failures are removed. In the same time, the needed durability of bridge is assured along further functional period with the same parameters of social needs.

b) Reconstruction caused by failures in carrying bridge superstructure can cause under specific conditions even such decision that can produce reduction of bridge usage (i. e. reduction of load carrying capacity), removing of substantial defects and leaving the bridge to serve to the end of the life time in limited operational conditions for some time only. Supposed period for this decision should not be more than 5 years.

Decision-making process about necessity or possibility, range and method of reconstruction is often very difficult because there is need to take into account set of many contradictorily acting factors namely above all:

- Purpose of reconstruction (including considerations about the social importance of the bridge).

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- Economical aspects (the viewpoint of moral life time), rate of initial costs to maintenance costs.

- Possibilities of technical solutions (comparison of structure condition with available technical means).

In decision-making process it can not be omitted the fact that the condition of bridges built in the same period, by the same technology and at the same traffic load, can be substantially different, namely through the climatic impacts and through the effects of aggressive environment. They were not envisaged in complex in the bridge design in the past, but they were solved only by the taking into account of code requirements that set down maximum permissible crack widths, eventually the necessary depth of cover layer.

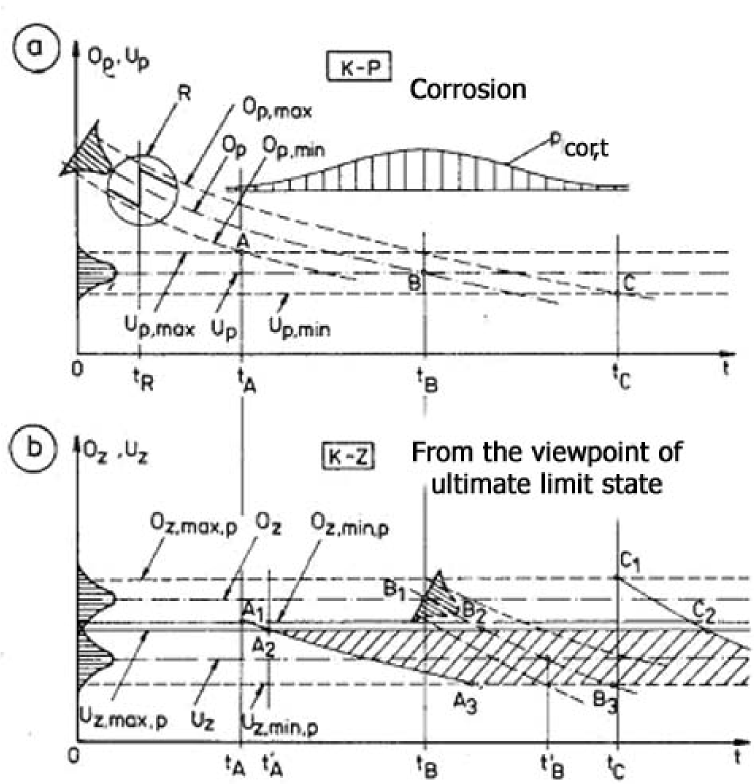

Fig. 2 Scheme of interaction of reliability system (structure, loads & environment in time t)

Both the effects of environment and the structure resistance against them are variable values, see [2], that can be influenced by many other factors both in

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project and in bridge construction and during its operation (e. g. by the shape of carrying bridge superstructure, surface structure, compactness of concrete, usage and arrangement of reinforcement, total level of production and its control, influences caused by shrinkage, strain, flow of interacting media, maintenance of operational devices).

On Fig. 2 the interaction of system (structure, loads and environment) is illustrated in simplified way during the time for the bridge that would demonstrate a long life time in favourable environment.

Fig. 2a shows the influence of random variability of environment impacts U_p and the influence of structure resistance O_p on the possible moment of the beginning of corrosion of concrete or reinforcement (degradation O_p by influence of U_p).

Fig. 2b further shows the course of reducing of the bridge structure resistance O_z from the viewpoint of ultimate limit state and eventual rise of failures from this viewpoint, which will come into being if $O_z < U_z$. It is evident that first failures can arise already much earlier than it is supposed ($t'_a < t'_b$), even though with the little probability.

Points marked out on Fig.2 (a, b) $A(t_a)$, $B(t_b)$, $C(t_c)$ show examples of possible moments of the beginning of corrosion and the $C_1 - C_3$ courses of reducing of the bridge structure resistance O_z against loading by the corrosion influence are connected with them.

From above mentioned examination it is evident that reliability and durability of bridge structure is complicated matter. There is need to proceed during the examination in a complex way and carry out always substantial analysis of all the possible causes under them it can come to the bridge failures. Only then, on the basis of complex analysis carried out in this way, we can come up to the decision-making process, how to proceed with bridge ahead. Even in a very careful approach, though for assessment and decision-making process we shall use results of diagnostic survey, eventually completed ones by loading test in any case we meet the certain risk.

4. RISK IN DECISION-MAKING PROCESS WHAT TO DO WITH THE BRIDGE IN A CERTAIN STAGE OF LIFE TIME

Each progress and each change bring the certain risk with themselves. We could understand it in connection with progress as a positive effect. We get information about risks that could be accurate, sometime less accurate and sometime even knowingly garbled. Risks remain often inexplicable and they produce fear and

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worries. There is need to assess the risks at construction and operation of bridge from various viewpoints, namely during the course of whole life time of bridges:

- collapse of carrying bridge superstructure can be an extreme case,
- total collapse of structure and its putting out of operation.

The risks in decision-making process, what to do with the bridge in a certain stage of life time, are important extraordinary. Just these risks should be the objective of other studies, because they can influence considerably the future of ever increasing number of built bridges.

5. NDT BRIDGE DIAGNOSTICS METHODS

Beside a classical bridge inspection methods, which are realized in regular periods (main, secondary, and control inspections) there are special inspections, which are carried out after disaster's occurrence, in case of detection of any change in soil body or after presence of rebar corrosion symptoms, etc.

The classical bridge inspection methods are visual inspection, carbonation depth survey, content of chloride survey, core analysis, etc.

New NDT methods like ground penetrating radar, ultrasound, acoustic emission, infrared thermography, radiography, vibration analysis are used more often today [3].

Bridge structure can be instrumented with structural testing system for medium and long-term monitoring. This way a structural movement or degradation over periods of time is monitored and the data can be downloaded via telephone modem and remotely viewed in real time or automatically stored on a periodic basis.

CDV- Transport Research Centre is dealing with two from above mentioned NDT methods (acoustic emission - AE and ground penetrating radar - GPR) [4].

CDV proposed the best practice guideline called Diagnostic survey of road bridges, monitoring methods and evaluation of rebar corrosion in concrete by acoustic emission method, which was submitted for an approval to the Czech Ministry of Transport.

Acoustic response of studied structure to an evoked loading is evaluated there.

Basically there are two possibilities for application of AE method. The first one is long term monitoring of a whole structure or its parts by array of AE sensors. The second one is one-shot investigation by AE method, which should be carried out periodically.

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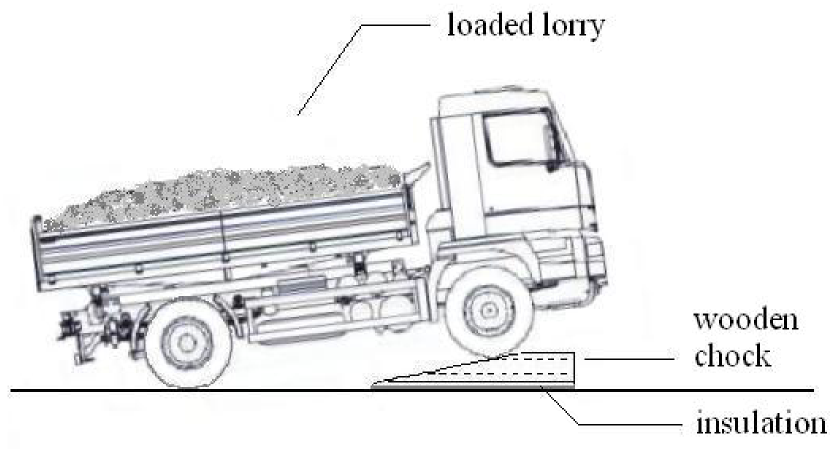


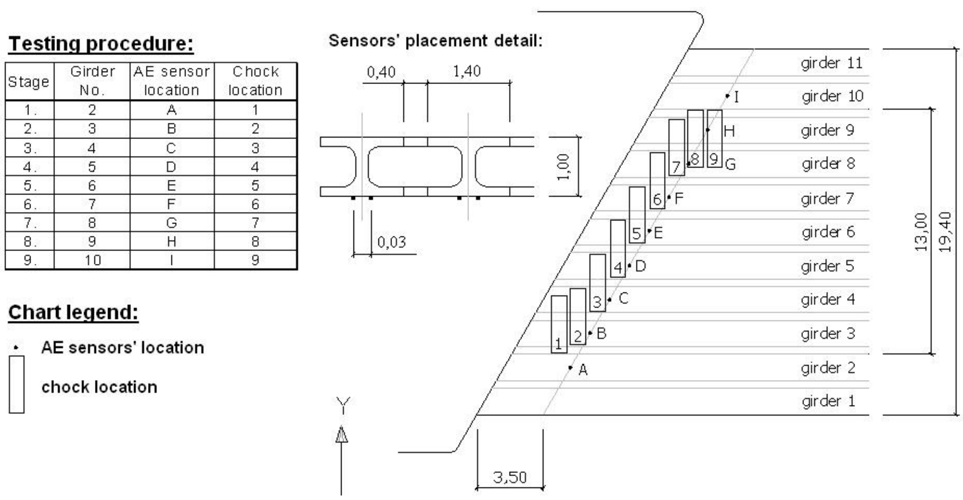
Fig. 3 Bridge dynamic loading with impact

In the proposal there are mentioned the following basic ways of a bridge loading:

static – loading vehicles are placed in the middle of the arch span,

dynamic without impact – loading vehicle or vehicles travelling along the bridge with constant speed 0,5 km/h,

dynamic with impact – loaded lorry travelling with constant speed across the wooden chock of constant high, see Fig. 3,
under traffic operation.



Stage	Girder No.	AE sensor location	Chock location
1.	2	A	1
2.	3	B	2
3.	4	C	3
4.	5	D	4
5.	6	E	5
6.	7	F	6
7.	8	G	7
8.	9	H	8
9.	10	I	9

Fig. 4 Bridge loading scheme

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The measurement is carried out on the base of prepared testing procedure, whose important part is a loading scheme. An example of loading scheme in case of dynamic loading with impact shows Fig. 4 (wooden chock placement, AE sensors location, etc.).

Recorded AE signals are analyzed (number of AE events and their parameters, frequency analysis, time-frequency analysis) and AE sources are classified.

Fig. 5 shows frequency spectra comparison of signals recorded during dynamic loading of the bridge with impact in the middle of the same girder before and after reconstruction of the bridge, where the original girders remained the same.

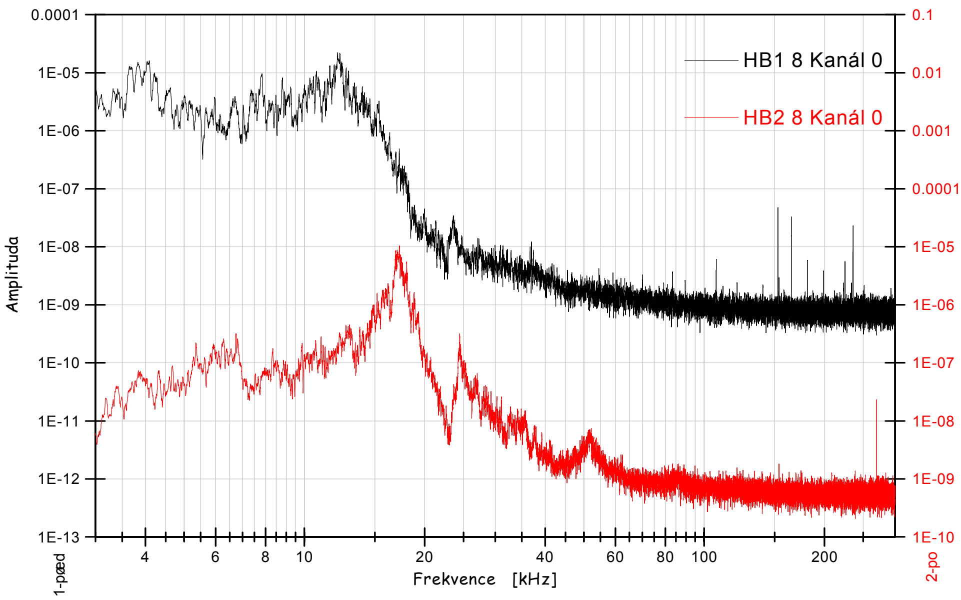

Fig 5 Girder evaluation: frequency spectra comparison (before and after reconstruction of the bridge)

CDV partakes on solving of some research projects where the Ground penetrating radar possibilities are studied in the field of road evaluation. Currently CDV concentrates its effort also to GPR usage in the bridge evaluation area [5].

In the Czech Republic the GPR diagnostics of bridges is limited to a pavement and a bridge deck control. The GPR problematic is not standardised in the Czech Republic in contrast to USA and UK.

Fig. 6 shows GPR radargram at the edge of Gutter bridge, which was subsequently reinforced and filled with concrete and its interpretation in the form of the graph.

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Usage of suitable combination of NDT methods gives more precise survey results. In the case of GPR it is appropriate combination with infrared thermography method.

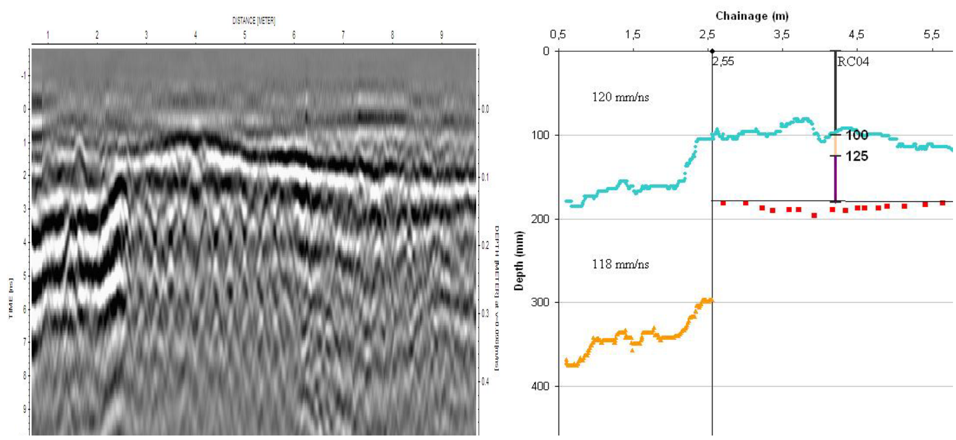

Fig. 6 Gutter bridge: GPR radargram, graph interpretation

6. CONCLUSIONS

The reliability and durability of the concrete and pre-stressed concrete bridges depend on their initial condition, on the quality of project and realization and further on the effects of operation and environment.

The durability of bridges is an indivisible part of their reliability. Important decisions are made just before the reconstruction of the bridge or after their failures.

In such decision-making processes some factors as purpose of reconstruction, economical aspects and possibilities of technical solutions and necessary risks too should have been taken into account.

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A performance grade of polymer - modified bitumen, according to SHRP specifications

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Summary

This paper present some results obtained on the evaluation of various bituminous binders used in rehabilitation, modernization and maintenance works, involving the SHRP/SUPERPAVE equipments and specifications, based on the performance criteria, in conjunction with the classical methods as those involving the thermal susceptibility, Fraass breaking point and chemical composition. The paper also includes recommendations concerning the improvement of the performance of these binders, by modification either with SBS or reactive polymers.

KEYWORDS: bitumen, performance grade, SBS polymer, reactive polymer, shear module, stiffness module.

1. INTRODUCTION

Bitumen is a visco-elastic material; its mechanical properties are influenced by time and temperature. During the service ability period, changes occur in the bitumen chemical composition, which lead to changes in the rheological behavior. In normal exploitation conditions, the behavior of the bitumen may be satisfying but in severe conditions the bitumen may suffer permanent deformations or cracks due to fatigue. In order to cope with the modern requirements, respectively obtaining a high plasticity limit, the bitumen can be modified by polymer addition.

Generally, polymers are used to modify bitumen for:
- increase the strength to permanent deformations (ruts) [1-6] by increasing the stiffness of the bitumen or by intensifying the elastic behavior;
- increase the strength to thermal and fatigue cracks [7];
- increase ageing and abrasion resistance;
- increase durability of asphalt pavement surface [2, 8].

Polyethylene, polypropylene, polystyrene and ethylene vinyl acetate copolymers are the main plastomers (synthetic polymers) studied for the modifying road binders; as for elastomers, the copolymers of styrene with butadiene are the best for modifying the bituminous binder [9]. These polymers are generally dispersed in the bitumen and if there isn't a good compatibility bitumen–polymer, a phases agglomeration may appear. To eliminate the problems of separation which may appear relative to storage stability of polymers modified bitumen, Du Pont Company produces an ethylene copolymer, respectively ethylene glycidyl acrylate (EGA), with the commercial name Elvaloy AM, which reacts easily chemically with bitumen and eliminates the problems regarding the phase separation [10]. This copolymer improves the properties of asphalt mixture at high temperatures, permanent deformation strength, susceptibility at temperature and also improves the creep compliance.

During the last decade, as in other European countries, the technical specifications regarding the road binders based on the performance criteria, elaborated within the American research program SHRP (Strategic Highway Research Program), produced a significant impact on the research directions as on the road technologies used in our country.

It is a well known fact that the main mechanisms which lead to deterioration of bituminous pavement surface are:

- permanent deformations and rutting phenomena at high exploitation temperatures, during summer time;
- fatigue phenomena under cycling load due to traffic, at normal temperatures;
- appearance of fissures on the road pavement surface at low exploitation temperatures, during winter time.

The SHRP specifications are able to evidence these mechanisms, as they are applicable to original bitumens as for modified bitumens with reactive or inert polymers.

Generally, most of the bituminous binders in initial state, regardless of their origin (made in /outside Romania) can't be considered proper for the high traffic roads. Consequently, specific research activities are initiated in our country, in order to improve the performance of bituminous binder requirements for the extremely severe climatic and traffic conditions specific to our national road network, using in this scope different polymers.

For this purpose, the bituminous binders having the penetration class D60/80 and D 80/100 have been modified with the SBS, respectively EGA polymers, evaluating their influence on the rheological characteristics and performance grade.

2. RESULTS AND DISCUSSIONS

The bitumen is a complex material, with a different behavior depending on temperature and time of loading (cyclic loads). At a high temperature and under long time loading bitumen behaves like a viscous liquid and flows (the response is instantaneous, without phase delay) but at low temperature and under short time loading, the bitumen behaves like an elastic solid (the response versus the loading is delayed with a 90° angle). At a corresponding temperature for service of most bituminous pavements under traffic, the bitumen behaves like a visco-elastic material, the delay between load and response is δ, from 0 to 90°.

Usually, up to their consistence, the road bitumens are divided in penetration classes (40/60, 60/80, 80/100). Since this classification isn't completely reflecting the viscous-elastic behavior of bituminous binder, a new classification methodology was established within SHRP, the performance grade (P.G.), depending on the maximum and minimum designed temperature for the road structures, considered as representative temperatures, as follows:

T max – the maximum designed temperature – represents the maximum average temperature measured in 7 days consecutively inside the road pavement, during the summer;

T min - the minimum designed temperature – represents the lowest temperature measured on the surface of the pavement, during the winter;

Example: PG: 64 - 28/the first number indicates the maximum temperature and the second indicate the lowest temperature at which the binder corresponds to the requirements taken into account during the design phase.

The parameters used for establishing the performance grade are: complex shear module $\mathbf{G}^*$, phase angle **δ**, stiffness module **S** and the slope **m**.

Two shapes of $\mathbf{G}^*$ and **δ** have been used [11]:
- the ratio $G^*/\sin\delta$ which characterizes the behavior of bitumen at high temperature and may be used to evaluate the permanent deformations;
- the product $G^*\sin\delta$ which characterizes the behavior of bitumen at intermediate temperature and may be used to evaluate the fatigue cracking;

According with SHRP specifications [11]:
- the value $G^*/\sin\delta$ must be greater than 1,0 KPa for original bitumen and 2,2 KPa for aged bitumen; for a good resistance to permanent deformation, it is advisable that the value $G^*/\sin\delta$ to be as large as possible (G* large values and sin δ small values);
- the value $G^*\sin\delta$ must be smaller than 5000 KPa. To ensure good fatigue strength to cracking, the bitumen should behave like an elastic material. The 5000 KPa value is established because both the viscous and elastic parts may become too high and the bitumen would no longer be able to effectively resist to fatigue cracking;
- the value of the stiffness module must be max. 300 MPa. If the stiffness is too large, the bitumen behaves like a fragile material, at low temperatures

cracking may appear. At low temperatures the values of stiffness module range are between 30 MPa and 300 MPa, so at the imposed value, 300 MPa the cracking phenomena is prevented.

- the slope **m** must have large values because at temperature changes and thermal efforts accumulation, the stiffness are changing relatively quickly and the bitumen may dissipate the efforts, such the cracking phenomena is avoided; SHRP specifies a minimum value of **m** at 0.3.

The values of the shear module and phase angle are determined for a temperature range between 46°…81°C and the values to the stiffness module and the slope between -6°…–40°C; the performance grade represents the corresponding temperatures at which the parameters simultaneously equals the imposed values. In the case of negative temperatures, -10°C must be added to the test temperature when the imposed value is reached. The corresponding temperature which fulfills the previously specified requirements varies, regardless of the type of the bitumen (bitumen in initial state or modified).

In the present paper, binders with penetration class D 60/80 and D 80/100 (made in/outside Romania) are divided on their performance grade according to SHRP methods (Table 1), with the aid of previously mentioned parameters. The Fraass breaking point (the minimum temperature corresponding to appearance of cracks for a thin bitumen film in bending) is compared to the temperature corresponding to the maximum value of stiffness module of 300 MPa.

The temperatures from Table 1 represent the testing temperatures at which the values of the determined parameters correspond to the technical conditions imposed by SHRP specifications.

From the analysis of data specified in the Table 1 it results that some of the local binders (B), or imported (C) are divided in performance grade totally inadequate to the specific conditions to the public road network in our country. The studied binders show extremely low performance at negative temperature, results obtained with the SHRP tests (results in bending for B and C sample) and for the classical tests (Fraas).At high temperatures similar results are obtained. The penetration index PI reveals that the analyzed samples show significant thermal susceptibility at positive temperatures during the hot season.

The penetration index PI is a measure of susceptibility of bitumen on temperature; the negative or positive values of PI indicate susceptibility at positive or respectively negative temperatures [9]. The practice shows that binders having PI values in the range of –1.0 <PI<+0.7 ensure a satisfying behavior of asphalt mixtures in exploitation.

Table 1: Evaluation of performance grade for indigenous (A, B) or imported (C) bitumen used in the road field

Source:	Technical Condition	A		B		C
Bitumen type		D60/80	D80/100	D60/80	D80/100	D60/80
Performance grade: PG X-Y		58 - 34	58 - 34	58 -16	52 - 16	58 -22
Breaking point Fraass, (°C)	max.-13	-23.7	- 24.1	- 16.5	- 15.1	- 10.1
Shear on original bitumen						
Temperature, (°C)	X	58	58	58	58	58
Ratio G*/sinδ, (kPa)	min.1.0	1.6	2.1	1.7	1.2	1.9
Shear on RTFOT[1] aged bitumen						
Temperature, (°C)	X	64	64	58	52	58
Ratio G*/sinδ, (kPa)	min.2.2	3.98	2.4	3.5	5.1	3.7
Shear on PAV[2] aged bitumen to 100°C						
Temperature, (°C)	-	19	13	22	19	22
Product G*x sinδ, (kPa)	max.5.000	1817.6	2271.1	4028.8	4793.1	4423.1
Bending on PAV aged bitumen to 100°C						
Temperature, (°C)	-Y	- 24	-24	-6	- 6	-12
Stiffness modulus S, (MPa)	max.300	181.96	161.13	108.3	101.2	287.1
Slope m	min.0.3	0.322	0.340	0.315	0.325	0.307
Penetration index IP, on original bitumen	-1<IP<+1	-0.83	-1.4	-0.98	-1.79	-1.75
Chemical composition (IATROSCAN) (%)						
Saturates	5…20	13.9	16.0	8.9	9.48	3.8
Aromatics	40…60	31.1	27.4	39.4	41.8	52.9
Resins	33…36	36.1	36.1	36.8	36.5	27.6
Asphaltenes	5…25	19.7	20.5	14.9	12.2	15.7
Coloidal instability index, Ic	max. 0.5	0.5	0.57	0.31	0.28	0.24

1 – rolling thin film oven test; 2- pressure aging vessel

In order to improve the qualities of these binders and getting them to the desired performance grade, researches were initiated regarding the use of different modifiers.

For improving the qualities of bitumen in initial state for the climate conditions in our country the modifiers should fulfill more conditions: improve the performances at high and low temperatures, ensure the stability of binder-modifier mixture during its stocking and preparation of the asphalt mixture.

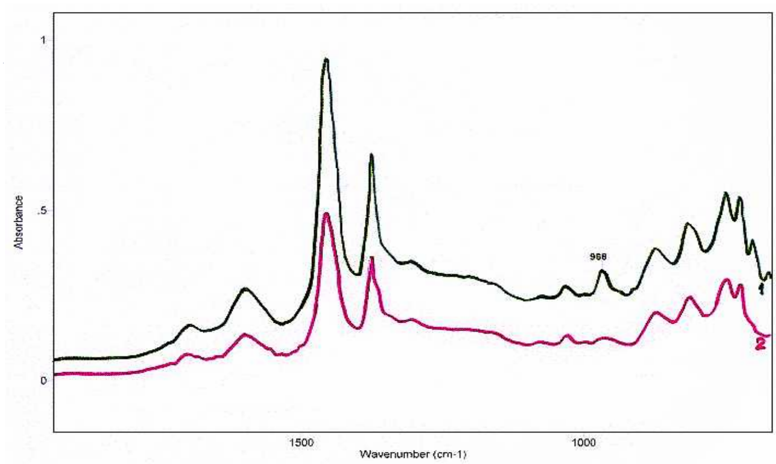
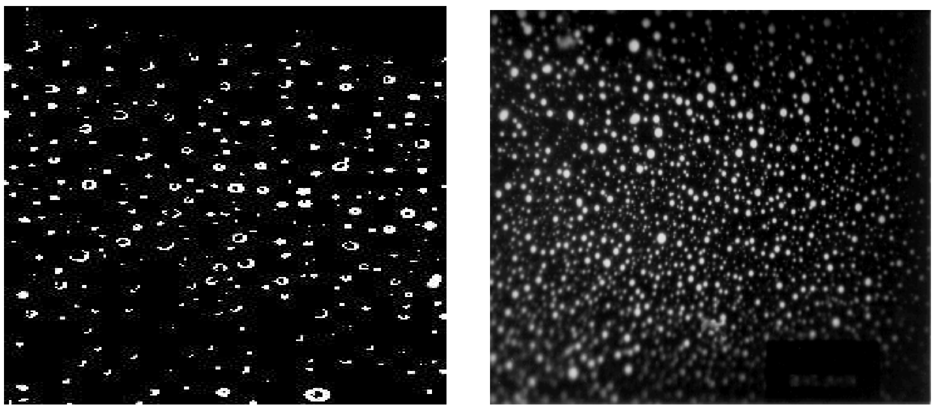


Fig. 1 – IR spectra of the bitumen in initial state (2) and SBS polymer modified bitumen (1)



(a) (b)

Fig.2 –Bitumen A 60/80 (a) and bitumen B 60/80 (b) modified with SBS polymer

For this purpose polymer SBS, respectively reactive polymer EGA – Elvaloy AM were used. The polymer was added as grains/powder in the preheated bitumen mass (175° - 180°C); the mass is continuously mixed with a speed to 250 rotations/min for 2,5 hours.

In the case of SBS the modification was evidenced by recording the IR spectra (Fig. 1) of original and modified bitumen (the absorption strip corresponding to the 965± 5 nm wavelength and a vibration of =C-H bond outside the plane of 1,4 trans butadiene in SBS, the missing strip in the unmodified binder) and also by

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fluorescent microscopy, evaluating the polymer dispersion degree within the bitumen (Fig. 2).

Notice that in the case of bitumen having an increased aromatic content (B), and accordingly a lower colloidal stability index (increased bitumen-polymer compatibility), the dispersion of the polymer within the bitumen is better (Fig. 2, b).

The evaluation of performance grade of SBS and Elvaloy AM modified bitumens is shown in Tables 2, 3.

Table 2: Evaluation of performance grade for indigenous bitumen A and modified with SBS polymer B bitumens

Source Bitumen type	Technical Condition	**A D 60/80**		**A D 80/100**	**B D 60/80**
SBS polymer content		+4%	+5%	+4%	+4%
Performance grade, PG X-Y		**76-34**	**76-34**	**76-34**	**70-28**
Breaking point Fraass, (°C)	-	-24	-24	-24	-18
Shear on original bitumen					
Temperature (1), (°C)	**X**	**76**	**76**	**76**	**70**
Ratio G*/sinδ, (kPa)	min. 1	1.47	1.50	1.30	2
Shear on RTFOT aged bitumen					
Temperature(1), (°C)	**X**	**76**	**76**	**76**	**70**
Ratio G*/sinδ, (kPa)	min.2.2	2.8	3.4	2.50	4.3
Shear on PAV aged bitumen to 100°C					
Temperature(1), (°C)	-	**13**	**13**	**13**	**19**
Product G*x sinδ, kPa)	max.5000	3769	2843	3425	4800
Bending on PAV aged bitumen to 100°C					
Temperature (1) , (°C)	**Y**	**-24**	**-24**	**-24**	**-18**
Stiffness modulus S, (MPa)	max.300	178	121	168	280
Slope m	min.0.3	0.367	0.367	0.300	0.320

1. (1) Temperature of testing for reached corresponding value of impose technical condition.

On the basis of the results shown in Table 2 it is observed that the temperature at which the corresponding technical SHRP parameters are fulfilled is improved by using SBS polymer. For the bitumen sample at initial state the ratio $G^*/\sin\delta$ (the elastic component) reaches the imposed value at 58°C; in the case of modified bitumen, the corresponding temperature for reaching the imposed value is 76°C for A bitumen and 70°C for B bitumen; the performance grade at higher temperature increases for all the bitumen types (from PG 58 to PG 76, in the case of A bitumen and from PG 58 to PG 70 in the case of B bitumen) (Fig.3 and 4).

In the case of SBS polymer modified bitumen its behavior improves at high temperatures, the elastic component improves the strength to permanent deformations. By modification with polymer the viscous component, $G^*\sin\,\delta$, increase for a given temperature, this meaning a lower fatigue strength. Regarding the behavior at low temperature, the Fraas breaking point is improved in the case of indigenous bitumen B (from –16.5°C to -18°C) and practically remains constant for the A bitumen (-24°C).

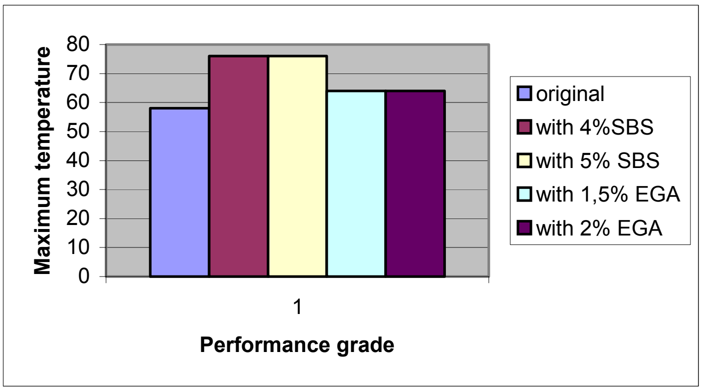

Fig.3 – Original and modified indigenous bitumen type A60/80. Performance grade to high temperature

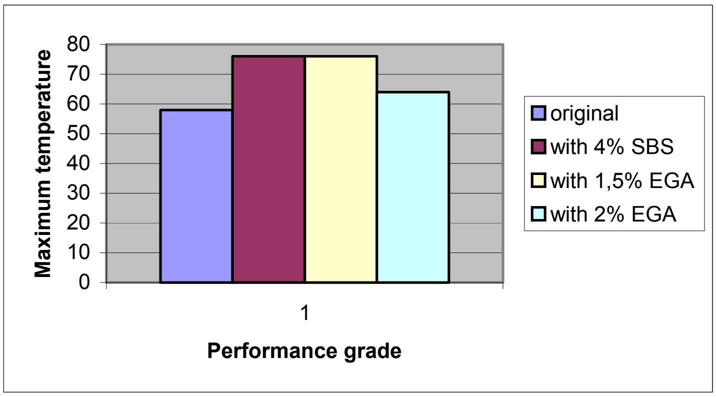

Fig.4 – Original and modified indigenous bitumens type A80/100. Performance grade to high temperature

This fact is correlated to the results obtained from bending, the temperature corresponding to the SHRP imposed value of stiffness module is -18°C in the case of B bitumen, and -24°C for A bitumen. So, adding the polymer improves also the performance grade at low temperature in the case of B bitumen (from PG -16 to PG -28).

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Even though the modification with polymer does not improve the performance grade at low temperature for all the studied bitumens, the SBS modified bitumen versus the original one show a lower stiffness; the polymer improves the relaxation of bitumen under effort.

Table 3: Evaluation of the performance grade for indigenous bitumen A/B and imported bitumen C modified with reactive polymer Elvaloy AM

Source Bitumen type	Technical condition	A D 60/80		A D 80/100		C D 50/70
Reactive polymer content		1.5 %	2 %	1.5 %	1.75%	1.5%
Performance grade: PG X-Y		64-34	64-34	76 -34	64 -34	64 -22
Breaking point Fraass, (°C)	-	-27	-27.5	- 28	- 29	- 17.3
Shear on original bitumen						
Temperature (1), (°C)	X	64	64	76	64	64
Ratio G*/sinδ, (kPa)	min.1.0	1.7	1.8	1.2	1.9	1.2
Shear on RTFOT aged bitumen						
Change of mass, (%)	-	0.186	0.148	0.242	0.236	+ 0.112
Temperature (1), (°C)	X	64	64	76	70	64
Ratio G*/ sinδ, (kPa)	min.2.2	4.3	4.5	4.0	3.7	2.4
Shear on PAV aged bitumen to 100°C						
Temperature (1), (°C)	-	13	13	19	16	19
Source Bitumen type	Technical condition	A D 60/80		A D 80/100		C D 50/70
Product G*x sinδ, (kPa)	max.5.000	3530	3363	2156.4	2156.4	4546.9
Bending on PAV aged bitumen to 100°C						
Temperature (1), (°C)	-Y	-24	-24	-30	-24	-12
Stiffness modulus S, (Mpa)	max.300	236.5	243.3	288.5	137.5	299.0
Slope m	min.0.3	0.361	0.326	0.347	0.381	0.304
Penetration index PI, original bitumen	-1<PI<+1	+0.3	-0.42	+ 0.57	-0.62	-0.2
Chemical composition (IATROSCAN) (%)						
Saturates	5…20			8.3	21.8	3.5
Aromatics	40…60			37.0	19.2	52.1
Resin	33…36			26.2	30.6	27.7
Asphaltene	5…25			28.5	28.4	16.6
Coloidal instability index, Ic	max.0.5			0.6	0.94	0.25

(1) – Temperature of testing for reached corresponding value of imposed technical condition.

From the results shown in Table 3, we notice that using reactive polymers for modifying the bitumen at initial state gives the advantage of a simpler technology; this implies a smaller polymer content (1…2;%); the polymer reacts with

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asphaltene from the bitumen at initial state and the result is a stable combination, bringing the bitumen to the desired performance grade and also improving the thermal susceptibility (improved penetration index).

Modified bitumen with reactive polymer shows smaller values of ratio $G^*/\sin\delta$ corresponding to SHRP method, at higher temperatures than those corresponding to bitumen at initial state, this leads to improvement of performance grade at positive temperatures from PG: 58 to PG: 64 and PG: 70, in the case of indigenous bitumen, respectively from PG: 58 to PG: 64, in the case of imported bitumen. This classification shows that the bitumens modified with reactive polymer EGA versus bitumens in initial state, have a better strength to permanent deformations.

Even if Fraass breaking point is improved by adding the reactive polymer EGA (from -24°C to -27°C/-29°C, in the case of indigenous bitumen and from -10°C to -17°C, in the case of imported bitumen), the temperature for SHRP, the corresponding values of stiffness module, and the slope **m** remain the same in the conditions of an unimproved performance grade (PG: -34 for indigenous bitumen and PG: -22 for import bitumen). Even more, the stiffness module recorded at the classification temperature shows higher values for the modified bitumen, so the EGA reactive polymer shows a lower ability of improving the relaxation of bitumen under effort, in comparison to the SBS polymer.

3. CONCLUSIONS

SHRP specifications based on the rheologic parameters (parameters completely reflecting the viscous-elastic behavior of bitumen) determined for the climate and traffic conditions in our country, proved to be a very important instrument in the evaluation of the performances of indigenous and import bitumens and also in quantifying the effect of different techniques for improving their quality.

SHRP tests on unmodified and modified bitumen prove the increase of performance in the case of bitumen modification with polymer.

Bitumen modified with SBS and EGA polymer show improvement of the performance grade at high temperature, leading to a better strength to permanent deformations. Depending on the type of the bitumen base, these polymers also improve the performance grade at low temperature.

Use of bitumen modified with reactive polymer, compatible to all studied bitumen types implies simplified technology and reduced production cost.

The adequate bitumen, with the required performance grade, may be selected according to the different climate and traffic conditions in our country.

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The electrical simulation of the rheological behavior of the asphalt mixes

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Summary

The paper presents the research work undertaken during author's doctoral studies on the electrical simulation on the rheological behavior of the asphalt mixes.

KEYWORDS: asphalt mixtures, electrical simulation, rheological behavior, analogical models

1. INTRODUCTION/GENERAL VIEW

1.1. The rheological behavior of the asphaltic mixtures is of visco-elastoplastic nature.

1.2. Material models (similar, analogical and structural ones) are used to simulate the rheological behavior. The analogical, the mechanical, and the electrical ones (the last one has the advantages of being very simple and very operative) are the most used.

1.3. The analogical models allow a better separation between the elastic deformation and the viscous and plastic ones than the rheological equations (relationships which connect the tension values both to the specific deformations and their derivatives which change over a period of time)

1.4. The different domains of the asphaltic mixtures behavior are represented according to the deformation amplitude (γ) and the number of cyclic loadings (N) at a certain rate.

2. RHEOLOGICAL MODELS

2.1. We mention below some of the mechanical models used for research regarding the behavior of asphaltic mixtures, models for which we further present the electrical simulation:

a. The Bűrgers Model;

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b. The Gretz Model;

c. The Bingham Model.

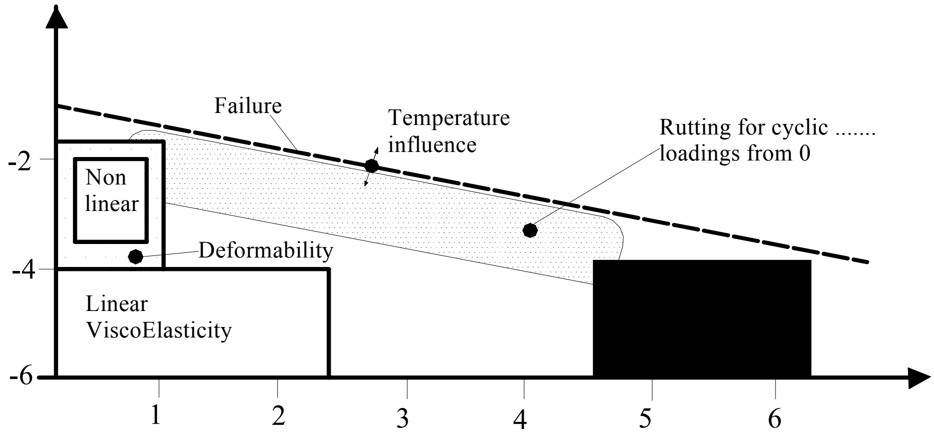

Figure 1.

2.2. The Bűrgers model consists of a Maxwell model and a Kelvin one, and models the instantaneous elasticity, the aftereffect elasticity and the viscous deformation properties.

The second figure presents the mechanical model (a) and the electric ones (c,d)

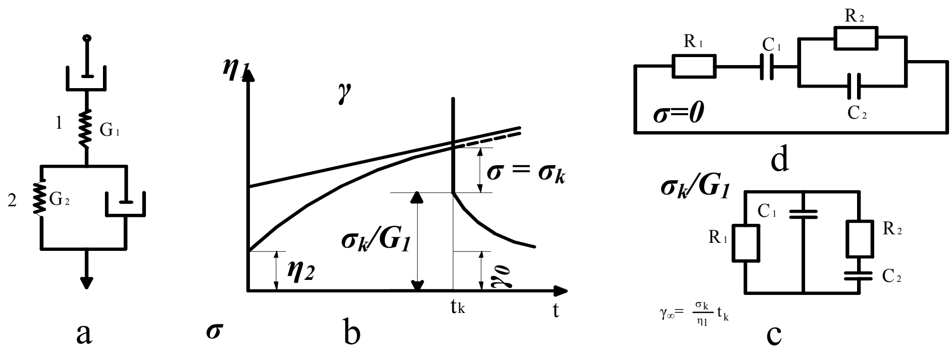

Figure 2.

G_1, G_2 – coefficients of elasticity;

η_1, η_2 – coefficients of viscuosity;

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c – the electrical model established through the analogy based on the condenser equation;

d – the electrical model established through the analogy based on the Ohm law;

c_1, c_2 –the capacities of the condensers;

R_1, R_2 –the electrical resistances.

2.3. The Bingham model (figure 3) shows the behavior of the asphalted mixture: elastic for small loadings and viscous after a certain value of the tension is overpassed.

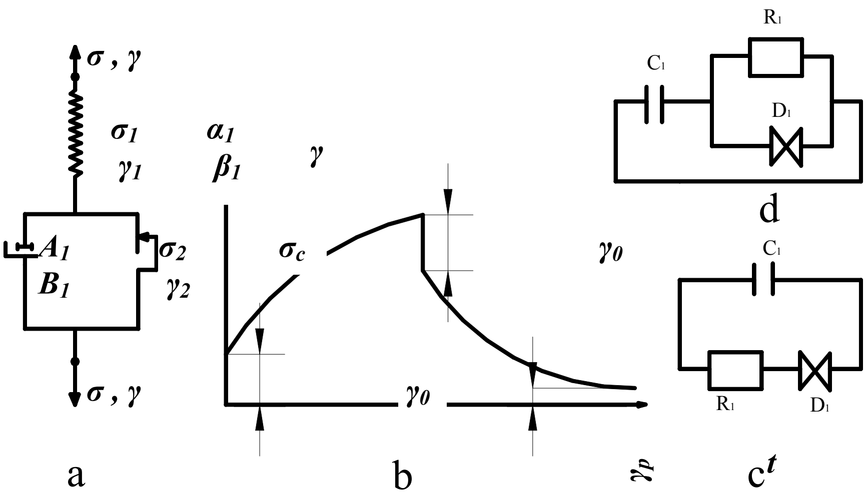

Figure 3.

2.4. The Gretz model is a non-linear model which presents both the reversible and the irreversible deformations.

Within the second element, the damper takes action only if the tension σ allows the overpassing of the friction force within the friction element''p''("p" has the same role as the diode from the electrical model).

The model we present is a non-linear one (from this point of view being similar to the Bingman one); therefore, the characteristic curve shows either the residual or the permanent deformation.

Figure 4 displays the mechanical model (a) and the electrical ones(c, d).

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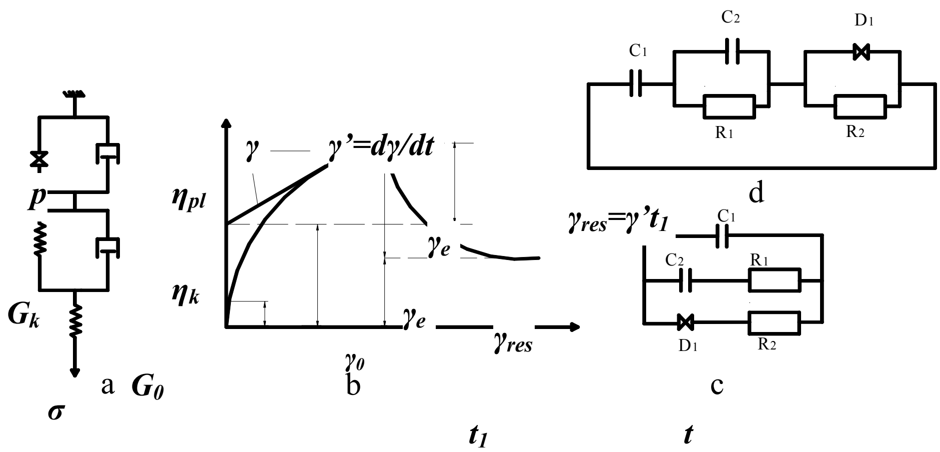

Figure 4.

The friction element ”p” is emphasized within the general rheological model displayed by figure 5.

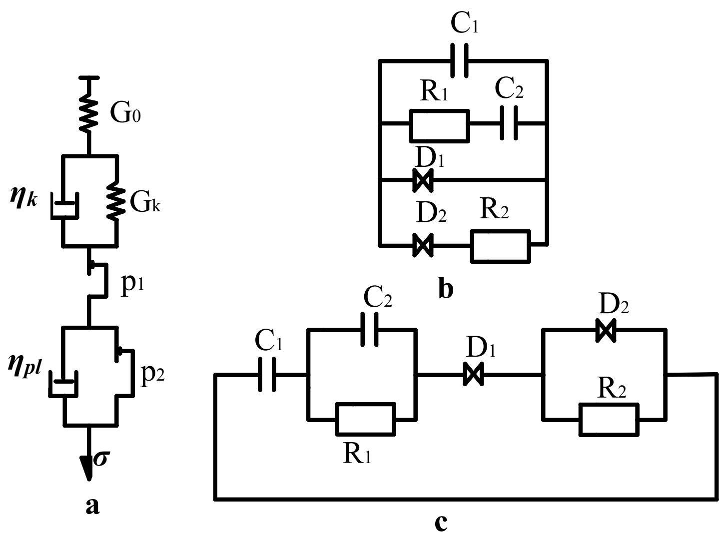

Figure 5.

The OrCad programme can be used to study the electrical models and it can simulate analogically, digitally or mixt (analogical-digital).

With the help of the running programme, the form of the signals is connected to time (being defined in time).

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The characteristic curve obtained through the electrical simulation and the characteristic curve obtained inside laboratory is being compared on specific sections (section connected to the elastic or the viscous behavior).

The trust level of the simulation has the form of statistic parameters represented by the correlation coefficient, the standard error and the maximum values of the positive/negative residuum.

3. CONCLUSIONS

The electrical simulation of the asphalted mixtures rheological behavior will be chosen over the mechanical modeling because of their simplicity and operativeness.

If we consider the electrical model, then the friction element model can be assimilated to a diode.

The simulation trust level will have the form of statistic parameter obtained from comparing the characteristic curves (the laboratory/modeled one) from specific section.

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Modelling an asphalt pavement in Portugal

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Summary

This article presents the modelling procedure of an existent pavement in Portugal, carried out by the author in the frame of "Leonardo da Vinci" Student Mobility Program, Contract RO/2004/PL93209/S, at Universidade do Minho - Center of Civil Engineering.

The 6 years old pavement under study exhibited an important extent of cracking and ravelling with high severity level, indicative of premature failure.

The assessment of the structural condition of the pavement requires the definition of its model. The adopted model is based on multilayer elastic (MLE) theory as it is one of most used models.

The establishment of the model comprised several tasks, such as: i) surface condition assessment, based on visual inspection; ii) coring in and out of the wheel path and over cracks; iii) deflection measurement by means of a falling weight deflectometer; iv) definition of homogeneous subsections; v) back calculation of the stiffness moduli using Bisar 3.0 Program; vi) temperature correction.

The back calculation of the stiffness modulus presented some difficulties as far as curve fitting is concerned. This might have been a consequence of using simplified models. Therefore, further research should focus this topic.

KEYWORDS: pavement modelling, stiffness module, homogeneous subsection, deflections.

1. INTRODUCTION

The tools available for modelling at the present time are wide. Despite that, simple models, such as multi-layered elastic models, are preferred, although asphalt pavements behaviour is viscoelastic. Enhanced models require data which is difficult to obtain and they are often time consuming.

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The layered elastic theory is credited to V.J. Boussinesq, who published his work in 1885 [1]. Boussinesq's multi-layered elastic model assumes that each pavement structural layer is homogeneous, isotropic, and linearly elastic.

The basic assumptions of these relatively simple mathematical models are: pavement layers extend infinitely in the horizontal direction, the bottom layer (usually the subgrade) extends infinitely downward and materials are not stressed beyond their elastic ranges.

A layered elastic model requires a minimum number of inputs to adequately characterize a pavement structure and its response to loading, which are: material properties of each layer (stiffness modulus, Poisson ratio), pavement layer thicknesses and interface, loading conditions. The outputs are the stresses, strains, and deflections in the pavement [1].

Bousinesq's model is currently used for designing new pavements and overlays. It is also used for assessing the structural condition of an existent pavement by back calculating the stiffness modulus of each layer. The back calculation procedure is based on comparisons between deflections calculated using a computer program and deflections measured on the pavement under evaluation.

One of the widely used computer programs for modelling flexible pavement systems is Elmod. Elmod is a component of a number of integrated software packages available from Dynatest for effective and efficient analysis and management of pavements. Elmod forms the core module of the Dynatest suite analytical programs [2].

In the United States, Washington State DOT has developed the Everseries Pavement Analysis Program. Everseries contains three independent programs for layered elastic analysis (Everstress), FWD pavement modulus back calculation (Evercalc) and flexible pavement overlay design (Everpave) [1].

Shell International Petroleum Company has developed the Bisar program for stress and strain calculations in asphalt pavement models [3]. The Shell methodology is widely used for modelling flexible pavements behaviour.

If the main issue is to calculate the stiffness modulus for the existent asphalt layers when modelling a pavement, than a back calculation procedure is required. COST Project 336 [4] presents a procedure fallowed by many institutions.

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In this work the guidelines presented in the COST Project 336 have been fallowed in order to establish the model of the national road "EN 206 Variant" between Carreira and Guimarães, in Portugal, aiming at assessing its structural condition.

The model definition comprised the following steps, developed on next sections: i) surface condition assessment; ii) coring; iii) deflection measurement; iv) definition of homogeneous subsections; v) back calculation of the stiffness moduli; vi) temperature correction.

2. ROAD AND PAVEMENT GEOMETRY

The EN 206 Variant between Carreira and Guimarães, shown in Figure 1, is 2 km long and it is constituted by 2 lanes per direction (3.5 m each), a 3 m separation between carriageways, 2 service lanes (2.5 m each), and shoulders (1 m each). The current cross-section has a transversal slope of 2.5 % to both sides.

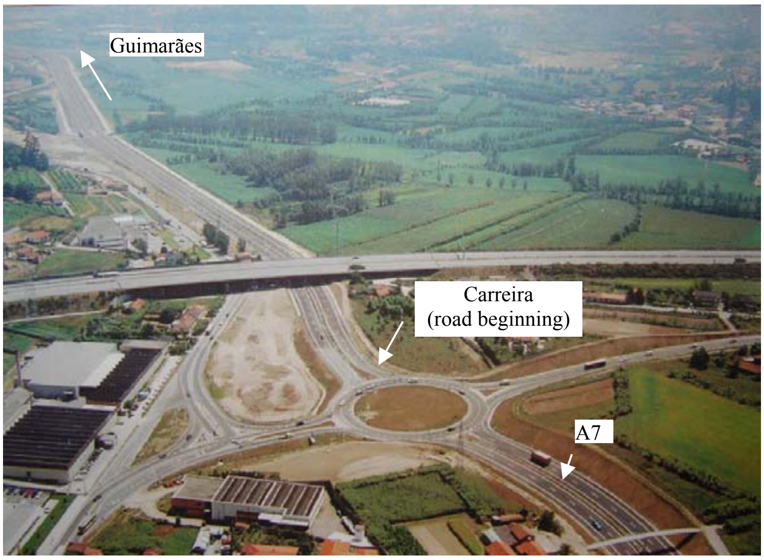

Figure1. General view of the EN 206 Variant

The design structure of the pavement is shown in Figure 2. It is constituted by 3 asphalt layers (wearing course – 6 cm; binder course – 6 cm; base layer – 12 cm) and an unbound sub base (graded aggregates – 20 cm).

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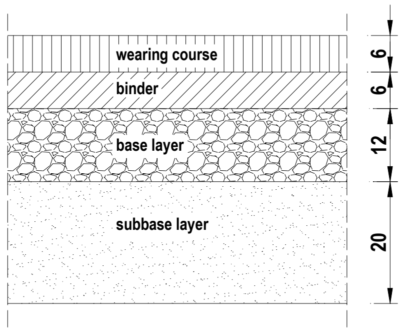

Figure2. Pavement structure

3. SURFACE CONDITION

The assessment of the surface condition was performed through visual inspection. The main distresses recorded were ravelling and cracking.

It was observed that, in general, the exterior lanes are more distressed than the interior ones. A different behaviour between driving directions was registered. In the direction from Guimarães to Carreira, the condition of the pavement is rather homogenous and better compared to the other direction. A small increase of the distress severity is recorded near the A7 roundabout, probably due to the high tangential forces as a result of breaking.

On the Carreira–Guimarães direction, two homogenous stretches can be established regarding the distress severity. The most distressed one is comprised in the first 700 m of the analyzed length and exhibits the highest distress severity and extent, if compared to the other stretches.

The ravelling observed on the surface of the road exhibits different levels of severity (Figure 3a). Some possible causes for the appearance of ravelling are: loss of bond between the aggregate particles and the asphalt binder; aggregate segregation; inadequate compaction during construction; mechanical dislodging by certain types of traffic such as vehicles with studded tires.
The cracking observed also exhibits different levels of severity as shown in Figure 3b.

Under repeated loading, longitudinal cracks begin to form at the base of the asphalt and propagate upwards (usually in the wheel paths). Then, these longitudinal

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cracks connect forming many-sided sharp-angled pieces that develop into a pattern resembling the back of an alligator or crocodile (alligator cracking). This leads to roughness, indicator of structural failure, cracks allow moisture infiltration into the base and subgrade and eventually results in potholes and pavement disintegration if not treated.

a) b)

Figure 3. Distresses on EN 206 VARIANT: a) ravelling; b) cracking

Alligator cracking occurs when there is an inadequate structural support for the given loading, which can be caused by many factors. Some of the most common are: decrease in pavement load supporting characteristics; stripping on the bottom of the hot mix asphalt layer; increase in loading (the pavement might loaded more heavily than anticipated in design); inadequate structural design; poor construction. Longitudinal cracking, transversal cracking and potholes were also found. These distresses are clearly the result of poor construction.

4. CORING

In order to investigate the causes of the early distresses observed, two slabs and several cores were extracted. One slab was extracted from an area exhibiting high severity distress and the other one in an area without distresses, located respectively at the wheel path on the direction Carreira – Guimarães at PK 0,26 km and at the shoulder on direction Guimarães – Carreira at PK 0,60 km. The cores were extracted all over the road, including on cracks.

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Coring allowed for checking design thickness, assessing construction quality and cracking direction.

It was found that layers thickness is similar to the one assumed in the design, nevertheless a severe reduction of the thickness near the right wheel path towards the shoulder was observed. This might partly explain the distress observed on the right lane. In addition to that, poor quality of top layers (wearing and binder coarse) was observed (Figure 4 a).

As a consequence of the surface poor quality, cracking started at the top of the asphalt layer and progressed downwards. This was stated in all cores extracted over cracks (Figure 4 b).

a) b)

Figure 4. Coring: a) splitting of the asphalt layer; b) top-down cracking

5. HOMOGENEOUS SUBSECTIONS

The homogeneous subsections have been established using the deflection measured with the FWD under the loading plate, every 20 m. 4 subsections with similar deflection were found. For each subsection a representative deflection bowl was selected. The representative deflection bowl is the one measured which best fits the 85 percentile theoretical deflection bowl calculated over all deflections of each homogeneous subsection. Table 3 shows the location of the 4 selected

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homogeneous subsections and Table 4 and Figure 5 present the representative deflection for each subsection.

Table 3. Kilometric position of homogeneous subsections

Carreira - Guimarães		Guimarães - Carreira	
A	0 → 1+340 m	C	0 → 0+640 m
B	1+340 → 1+980 m	D	0+640 → 1+980 m

Table 4. Representative deflections on each homogeneous subsection

	Sensors [mm]	0	300	450	600	900	1200	1500	1800	2100
A	average	323,6	262,5	225,8	192,9	132,5	92,3	63,3	46,6	35,3
	standard deviation	70,4	53,8	44,9	37,6	25,9	18,3	12,5	9,1	6,9
	rep. deflection (85%)	**396,5**	**318,3**	**272,3**	**231,8**	**159,4**	**111,3**	**76,3**	**56,0**	**42,4**
B	average	386,7	311,0	266,8	228,6	157,9	111,1	76,5	56,7	42,3
	standard deviation	98,1	68,2	54,7	43,2	27,1	17,9	11,6	8,6	6,7
	rep. deflection (85%)	**488,3**	**381,7**	**323,6**	**273,3**	**186,0**	**129,7**	**88,6**	**65,6**	**49,3**
C	average	554,2	427,9	354,1	288,2	182,9	119,0	78,7	58,1	42,9
	standard deviation	171,9	122,6	93,7	65,1	34,0	19,2	12,0	7,9	5,9
	rep. deflection (85%)	**732,4**	**555,0**	**451,2**	**355,7**	**218,2**	**138,9**	**91,1**	**66,3**	**48,9**
D	average	391,9	302,9	254,7	213,5	138,5	93,2	62,1	44,6	33,3
	standard deviation	130,0	81,4	60,9	45,0	21,7	13,1	7,2	5,0	3,7
	rep. deflection (85%)	**526,6**	**387,3**	**317,8**	**260,1**	**161,0**	**106,8**	**69,6**	**49,8**	**37,1**

These results correlate with the surface condition assessment: Section A, (Guimarães - Carreira, PK 0+000 to 1+340) presented less distress than the other sections, and also presented the smaller deflections.

Section C (Carreira - Guimarães, PK 0+000 to 0+640), the most damaged section, presented the highest deflection which is nearly twice the deflection of section A. Deflection on sections B and D do not seem significantly different and they are slightly higher than in section A.

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6. BACKCALCULATION OF STIFFNESS MODULI FROM FWD SURVEY

In order to calculate the stiffness modulus for each layer, the representative deflections of the homogeneous subsections were used as input data into Elmod 5 software. Initial stiffness moduli were obtained with this procedure.

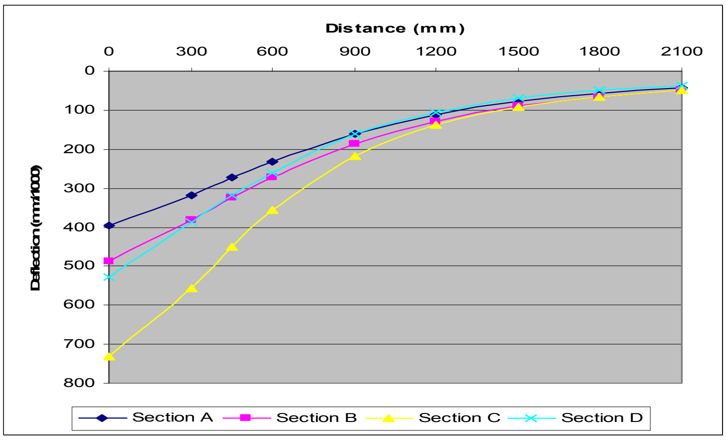

Figure 4 - Representative deflection bowls for each road section

Later the stiffness moduli were optimized by using Bisar3.0 software. A stiff layer with a modulus of 1500 MPa was introduced 2.0 m below the top of the sub grade in order to increase accuracy and to simulate the non-linear behaviour of the sub grade [5]. In Table 5 the adopted thicknesses for each layer as well as the stiffness modulus obtained from Bisar 3.0, which led to deflections that matched the representative deflections bowls for each section can be found. In Table 6 the corresponding calculated deflections are presented.

Table 5. Stiffness moduli for each section

Layer	Thickness (m)	Poisson's Ratio	Stiffness modulus (MPa)			
			Section A	Section B	Section C	Section D
Asphalt	0,24	0,35	5000	3700	2100	2700
Sub-base	0,20	0,40	220	220	100	165
Subgrade	2,00	0,45	60	50	46	62
Stiff layer	-	0,35	1500	1500	1500	1500

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Table 6. Calculated deflections

Distance from loading (m)	Deflection (µm)			
	Section A	Section B	Section C	Section D
0,0	395,4	485,8	720,0	518,1
0,3	316,2	383,0	542,6	384,9
0,5	274,0	330,2	454,8	321,2
0,6	233,6	280,2	374,5	263,6
0,9	162,6	193,3	241,8	169,7
1,2	106,9	125,9	146,1	102,8
1,5	65,7	76,5	80,9	57,5
1,8	36,8	41,9	38,8	28,4
2,1	17,3	18,9	12,9	10,5

Figure 4 presents both the representative deflection bowl and the one obtained with BISAR for sections A to D.

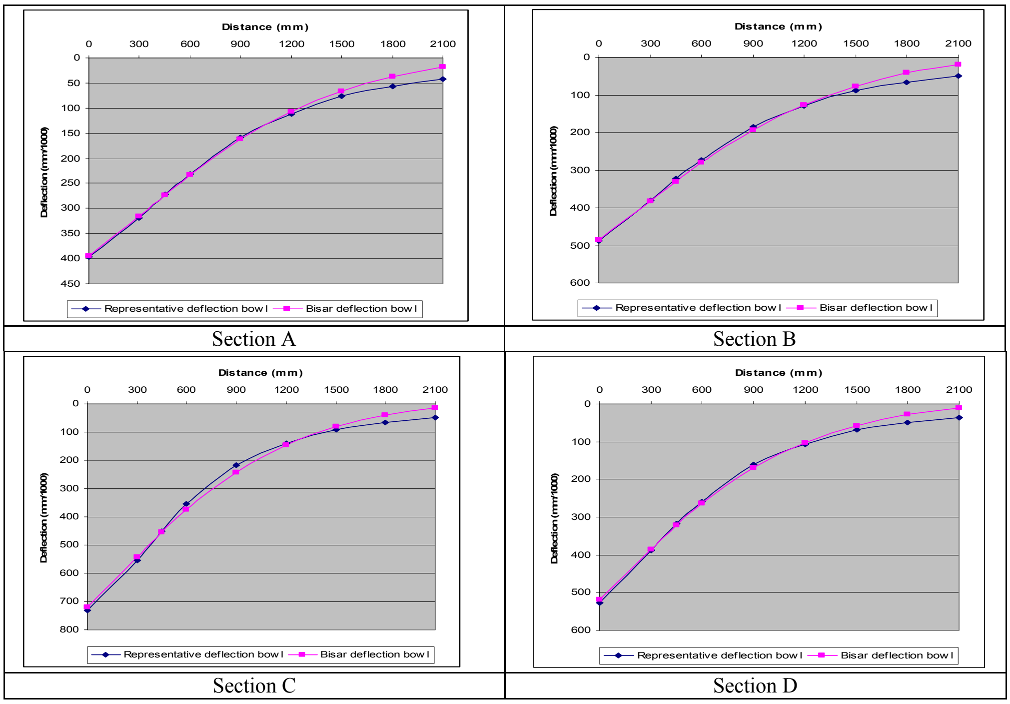

Figure 4. Measured and calculated deflection bowls

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As it can be observed in Figure 4, from the centre of the loading plate to 1200 mm a good match has been achieved. From 1200 mm to 2100 mm, the deviation between bowls increases with a higher value for the measured deflection. The end part of the deflection bowl is representative of the response of the subgrade to loading. This means that linear elastic programs are not reliable when the subgrade is concerned and back analysis procedure should be reviewed in order to take into account the effect of the increase of asphalt layers thickness on stress distribution.

7. TEMPERATURE CORRECTION

The stiffness modulus of asphalt layers is highly dependent on temperature. Comparisons can be made only if the moduli are determined for the same temperature. Taking this into account, the moduli calculated for each homogeneous section must be corrected in order to assess its evolution regarding design assumptions. The temperature at mead depth of the asphalt layer when the deflection was performed was nearly 16^0 C for all homogeneous sections. In view of the fact that the design temperature is $24{,}5^{\circ}$ C, a correction of the stiffness moduli is required. This correction has been made using Equation (1), which is suggested in several bibliographic sources [5].

$$\frac{E_{T1}}{E_{T2}} = \frac{1{,}635 - 0{,}0317 \cdot T_1}{1{,}635 - 0{,}0317 \cdot T_2} \tag{1}$$

In Table 7 the stiffness moduli for asphalt layers, after temperature corrections from 16ºC to 24,5ºC, are presented.

Table 7. Final stiffness moduli after temperature correction

Stiffness moduli (MPa) at 24,5ºC				
Design	Section A	Section B	Section C	Section D
3800	3805	2816	1598	2054
200	220	220	100	165
100	60	50	46	62

As expected, a great reduction on asphalt layers stiffness took place after 6 years under traffic loading in sections B, C and D. Sections C and D also show an important decrease of the stiffness of the unbound layer and subgrade. This means that the bearing capacity is significantly reduced.

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8. CONCLUSION

This paper is the result of the work performed at the “Universidade do Minho” - Center of Civil Engineering in the frame of “Leonardo da Vinci” Student Mobility Program, Contract RO/2004/PL93209/S. The work focused the structural assessment of the road “Variant to EN 206”, in Guimarães, Portugal, aiming at designing an appropriate overlay.

The structural assessment comprised several tasks such as: i. surface condition assessment; ii. coring; iii. deflection measurement; iv. definition of homogeneous subsections; v. back calculation of the stiffness moduli; vi. temperature correction.

It was found that ravelling and cracking are the main distresses, both with an important extent of high severity level. Poor quality of surface layers (wearing course and binder course) was highlighted by coring, what might have caused cracking which progressed from the top downwards. In addition, the analysis of the deflection measured with the FWD correlated with the distress severity levels observed on the surface and led to the definition of 4 homogeneous sections.

The back analysis procedure, based on the multilayer linear elastic analysis theory, has shown to be unsuitable for back calculating the subgrade modulus. This is indicative that results from linear elastic programs might not be reliable when the subgrade is concerned and that back analysis procedure should be reviewed.

Finally, as expected, it was found that surface modulus of sections B, C and D have reduced significantly after 6 years under loading. Further work should focus the causes of premature failure of the asphalt layers as well as a suitable approach for back calculation of subgrade stiffness modulus.

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Definition of homogenous road sectors according to COST 336

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Summary

The present paper presents the work undertaken by the author in the frame of "Leonardo da Vinci" Student Mobility Program, Contract RO/2004/PL93209/S, at Universidade do Minho - Center for Civil Engineering, in Portugal.

The work dealt with the structural assessment and design of the rehabilitation of a highway sector in northern Portugal. A brief description of the main activities performed and the main results of the work are presented. It included, surface and structural condition assessment, coring, laboratory testing, and estimation of residual life and overlay design.

A methodology to divide the road into homogenous sectors, according to recommendations of COST 336 Action Final Report, is applied in order to optimize the rehabilitation activities. The method is based on the computation of the cumulative sum of the deflection, on validation of the homogeneity and analysis of the statistical relevance of the division.

The methodology is easy to use and results, in this case, matched the field visual observations and the results of the laboratory tests. Furthermore, it could be easily integrated in a computer program.

KEYWORDS: Falling Weight Deflectometer, homogenous road sectors, coefficient of variance.

1. INTRODUCTION

One of the steps of rehabilitation design, both at project and at network level, is dividing the road section into homogeneous subsections. Data collection, storing and processing is important and expensive, so it is necessary to define road stretches on which they are to be retrieved and used with efficiency and reliability.

A homogeneous subsection is a part of the road in which the measured deflection bowls (or any other variable) have approximately the same magnitude and where it is not possible to subdivide it into subsections with significantly different behaviour [1].

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The subdivision is made not only for determining the stretches on which a certain rehabilitation method will be applied, but also to determine the necessary number and positions of coring location for further testing of the material. It can use a variety of manual and statistical techniques.

The importance of creating proper analysis sections cannot be overemphasised. Without appropriate sections, it is impossible to establish the correct investment decisions for the network. There are two stages for the sectioning process [2]: analysing the attributes of the road network and breaking it into sections and transforming the attributed data so that they adequately represent the road sections for the purposes of analysis.

For flexibility and reliability, it is recommended that road division into homogenous function is done automatically based on available data. McPherson and Bennett [3] recommend the use of an automatic sectioning function to create 'homogeneous' sections based on inventory and condition data.

This work particularly addresses the methodology recommended in COST Project 336 [1]. It integrates the procedure used to design the rehabilitation of the national road "EN 206 Variant" between Carreira and Guimarães, in Portugal. Next sections present a brief description and the main achievements of the work undertaken required to designing the overlay and a detailed description of the method applied to establish the homogeneous road sectors.

2. BRIEF DESCRIPTION AND MAIN ACHIEVEMENTS OF THE WORK

The work undertaken dealt with the study of a highway sector in northern Portugal, EN 206, between Guimarães and the A7 Highway distributor ring, a 3100m long road sector, the construction of which ended in 2000. A general view of this road sector is presented in Figure 1. The premature distress occurrence on it required a study of the possible causes of the abnormal situation and the proposal of viable rehabilitation solutions.

The study begins with general presentations on asphalt pavements and their behaviour, the general management and rehabilitation problems at project and network level, the distress types that can occur on flexible pavements and possible technical solutions.

2.1. Road description

The road description accounts for the geometrical characteristics of the longitudinal and transversal profile and for the initial pavement structure given in the design

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project: wearing course – asphalt mixture (6 cm); binder – asphalt mixture (6 cm); base layer – bituminous macadam (12 cm) and subbase – graded aggregates (20 cm).

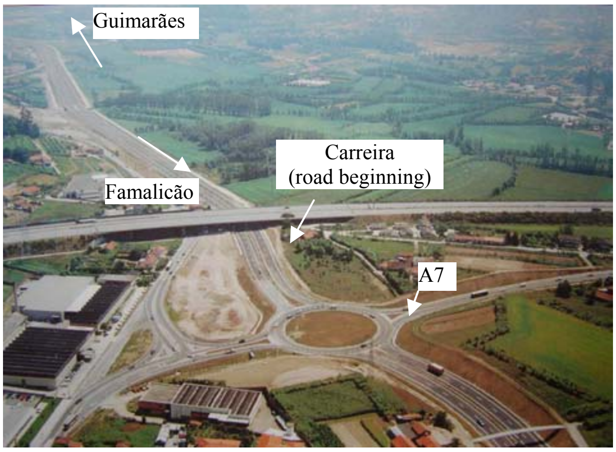

Figure 1. EN 206 Variant – view from the A6 distributor ring

Figure 2 presents the studied pavement system structure. According to the geotechnical study, the foundation soil is granite so the material extracted from the excavations was used for fillings. The soil is generally granite gravel or decomposed granites, with good mechanical characteristics. The quality of materials is attested through quality control performed during the construction works.

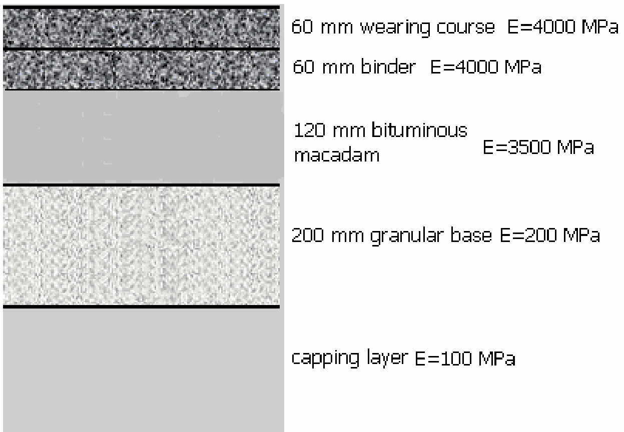

Figure 2. EN 206 Variant – pavement system

2.2. Traffic assessment

The traffic and temperature characteristics were re-evaluated for the initial project and for the present-day situation. A traffic counting was performed, what led to the conclusion that the actual traffic does not exceed the design traffic, eliminating then one of the possible hypotheses for the explanation of early distress occurrence.

2.3. Surface condition assessment

The surface condition assessment was one of the major problems encountered, as the main problems observed were ravelling and alligator cracking. After a brief description of all possible types of distress on flexible pavements, the project focuses on the quantification of these two distress types. The visual inspection and the methodology used led to the conclusion that, although ravelling is generalized and probably an indicator for top-to-bottom problem occurrence (poor wearing course material quality), there are compact sectors on which extended alligator cracking might show that the problem is actually the bearing capacity of the foundation.

Both the Portuguese distress catalogue [4] and the Pavement Condition Rating (PCR) methodology [5] were used in order to quantify in a global index the state of the surface, for each 10 meters sector along the length of the road. Figure 3 presents the result for one direction, both lanes, in terms of deduct points computed with PCR methodology (higher values means higher distress levels).

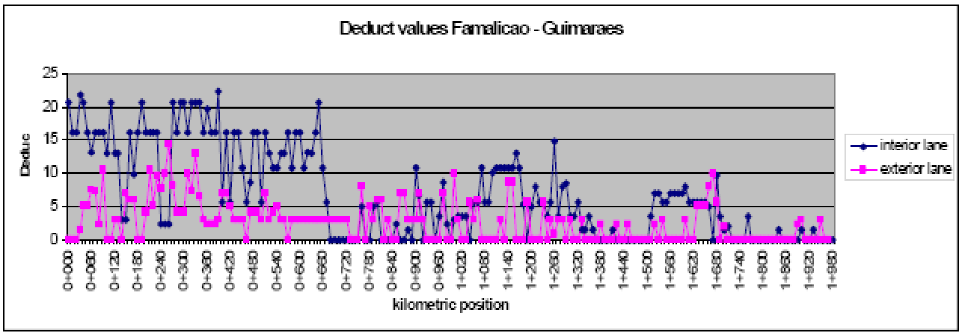

Figure 3. EN 206 Variant – PCR deduct values Famalicão - Guimarães

2.4. Structural condition assessment

The structural condition assessment was done using the falling weight deflectometer (FWD), presented in Figure 4. The studied road sector was divided into homogenous stretches based on the FWD results in correlation with the surface state assessment conclusions.

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Figure 4. Falling Weight Deflectometer

2.5. Coring

Coring was also performed to check for the direction of the cracks propagation. The visual inspection showed a bad condition of the wearing course, with top-to-bottom cracking or cracks that go to the whole depth of the asphalt layer. The bituminous macadam condition was usually good. In addition, it was noticed that the bonding between these two layers was weak; they could be easily separated during samples extraction.

2.6. Laboratory testing

The following laboratory tests were performed: grading – sieving method, water content by drying in a ventilated oven, sand equivalent test, methylene blue test, resistance to fragmentation by the Los Angeles test method, determination of the laboratory reference density and water content – Proctor compaction, California bearing ratio, stiffness moduli and fatigue life with the four points bending test, for bituminous layers. Their results generally followed the project specifications. The exception was the content of fine aggregates for the granular base and capping layer from the most damaged sector that was almost twice bigger than maximum admissible.

2.7. Residual life

Residual life was calculated from fatigue tests carried out on the extracted samples and estimated through design data.

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The number of residual standard axle loads calculated from fatigue tests is 5,7E+06 ESAL.

The design values for the fatigue life (N), as given in the project for 2005 and 2016, are respectively 11,7E6 and 29,3E6 ESALs.

The design residual life (N_{rez}) in 2005 was estimated by equation (1).

$$N_{rez} = N_{2016} - N_{2005} = 17{,}61E + 06\ \text{ESALs} \tag{1}$$

The comparison of both residual lives indicates that rehabilitation work might be required.

2.8. Overlay design

The overlay design was carried out after modelling the existent pavement. Modelling relied on deflection measurement with the FWD and back-calculation of stiffness moduli of the pavement layers. The SHELL design criteria were adopted – fatigue cracking of asphalt layers and structural deformation of unbound layer.

All homogenous stretches need structural rehabilitation, the cause relying in both week subgrade and highly damaged surface course. The most damaged stretches required 8 cm while the least distressed portions required only a thin overlay, which for technological reasons was considered 4 cm thick.

3. DIVISION OF A ROAD INTO HOMOGENOUS SECTORS ACCORDING TO COST 336

In order to analyze the present situation with regard to the surface state and structural ability of the pavement system, and to propose a rehabilitation solution, it was necessary to perform the division of the studied road into homogenous road sectors. This task was performed using the methodology proposed by *COST 336 Action Final Report, Cap. 4: FWD Project Level Guide* [1]. This methodology includes the following steps, which are developed in the next sections:

1. Input data: maximum deflections recorded with FWD on the studied road;
2. Computation of the mean maximum deformation;
3. Computation of the cumulative sums (Si);
4. Drawing the graphic (i/S_i);
5. Separating the zones for which the slope of the graph is approximately constant (a change in slope indicates inhomogeneity);
6. Testing the statistical relevance of the division: determination of the homogeneity level for each sector, using the coefficient of variance CV;

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7. Checking if there is a statistically significant difference between means of consecutive homogenous sectors by using Student's t-Test.

3.1. Subsection identification based on cumulative sum of variable

There are several statistical techniques available to divide a series of data into homogeneous parts. One of these techniques is the cumulative sum method. With plots of the cumulative sums of the deviations from the mean of the deflections against test point it is possible to discern these subsections. The cumulative sum is calculated in the following way:

$$S_1 = x_1 - x_m \tag{2}$$

$$S_2 = x_2 - x_m + S_1 \tag{3}$$

$$S_i = x_i - x_m + S_{i-1} \tag{4}$$

Where: x_i - deflection measured at test point i;

x_m - mean deflection of each main section;

S_i - cumulative sum of the deviations from the mean deflection at the test point i.

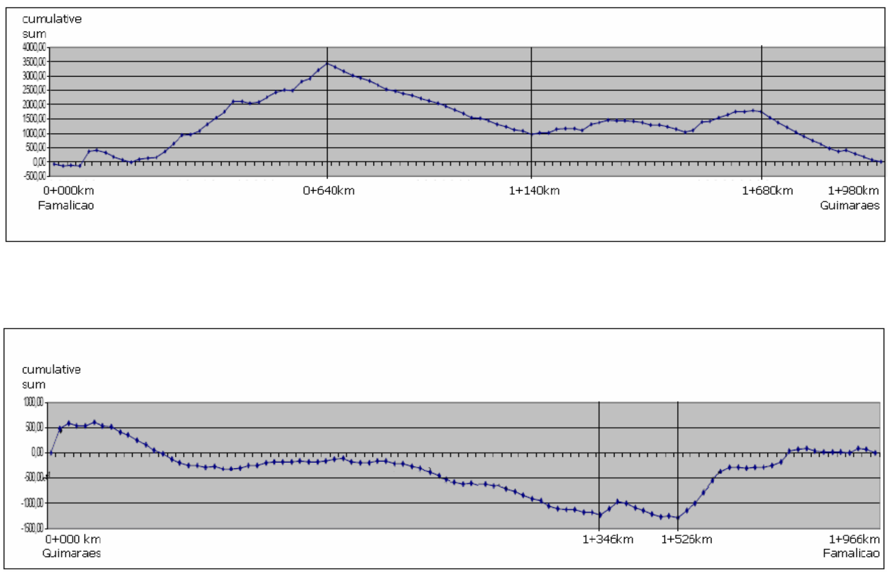

Figure 5. Separation of cumulative sums graphic into regions with almost constant slope

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Using the cumulative sums, the extent to which the measured deflections on a certain part of a road section are different from the mean deflection of the whole section can easily be determined. Changes in slope of the line connecting all cumulative sum values will indicate inhomogeneity [6].

The statistical analysis of the data using this method was performed and the resulting graphics are presented in Figure 5. The corresponding homogeneous road sectors are presented in Table 1.

Table 1. Homogenous road sectors based on cumulative sum analysis

Famalicão - Guimarães	
1	0 → 0+640 m
2	0+660 → 1+140 m
3	1+160 → 1+700 m
4	1+720 → 1+980 m
Guimarães - Famalicão	
1	0 → 1+366 m
2	1+366 → 1+526 m
3	1+526 → 1+966 m

3.2. Testing statistical significance of subdivision

For determining the level of homogeneity, one can make use of the coefficient of variation (CV). This parameter is defined as the ratio of the standard deviation over the mean value per section [6].

The mean value is defined as:

$$\bar{x} = \frac{1}{N} \cdot \sum_{i=1}^{N} x_i = \frac{x_1 + x_2 + ... + x_N}{N} \tag{5}$$

where: N - number of variables.

The standard deviation is defined as:

$$\sigma = \sqrt{\frac{1}{N} \cdot \sum_{i=1}^{N} \left(x_i - \bar{x}\right)^2} \tag{6}$$

The following list shows typical classes of CV:

CV < 20% - good homogeneity;

20% <= CV < 30% - moderate homogeneity;

30% <= CV < 40% - poor homogeneity;

CV >= 40% - inhomogeneity.

The CV is a measure of the consistency of the spatial variability measurements within the individual sections/subsections. Although the CV may indicate that a section or a subsection is not very homogeneous, it gives no indication of the possibility of subdividing it. CVs greater than 30% usually indicate a highly skewed distribution produced, for example, by a number of relatively "stiff" test points within a weaker subsection.

For the analyzed homogenous stretches, the homogeneity outcome is presented in Table 2.

Table 2. Statistical data for the homogenous road sectors

Famalicão – Guimarães			
standard deviation 1	156,1667	standard deviation 2	47,51218
mean 1	506,82	mean 2	313,95
CV 1 (%)	30,81281	CV 2 (%)	15,13358
standard deviation 3	99,10054	standard deviation 4	56,3189
mean 3	428,67	mean 4	290,59
CV 3 (%)	23,11818	CV 4 (%)	19,38071
Guimarães – Famalicão			
standard deviation 1	79,45857	standard deviation 2	68,70807
mean 1	293,46	mean 2	286,31
CV 1 (%)	27,07625	CV 2 (%)	23,99783
standard deviation 3	89,99587		
mean 3	360,06		
CV 3 (%)	24,99441		

Averages of deflections are calculated for each test line within each section. If the test lines are considered equal in terms of structure, materials, maintenance, edge support, etc., the averages are then tested for statistical significance of differences between the test lines. This can be done using the Student's t-test.

The variance of the difference between the two means (σ_d^2) and the value of the t-coefficient are:

$$\sigma_d^2 = \frac{\sigma_1^2}{n_1} + \frac{\sigma_2^2}{n_2} \tag{7}$$

$$t = \frac{\overline{x_1} - \overline{x_2}}{\sigma_d} \tag{8}$$

Tables 3 to 7 present these statistical data and the ones necessary to compute them, for each pair of consecutive homogenous road sectors. Once the t-value was

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computed, a risk level (alpha level) was set and the degree of freedom (the number of testing points on the consecutive tested road sectors minus 2) was determined. The computed t-value was compared to the values tabulated in a standard table of significance (Table 8) to determine whether the t-value is large enough to be significant. In Table 8, at (n_1 + n_2 -2) degrees of freedom (DOF) it is chosen the level of significance required (p = 0,05) and read the tabulated t value. If the calculated t value exceeds the tabulated value then the means are significantly different.

Table 3. Statistical data and t value for stretches 1 and 2, Famalicão - Guimarães

$n_1 = 34$	$n_2 = 24$	(σ_d^2)	811,35
$\bar{x}_1 = 506{,}82$	$\bar{x}_2 = 313{,}95$	(σ_d)	28,48
$\sigma_1 = 156{,}17$	$\sigma_2 = 47{,}51$	t	6,77
$\sigma_{12} = 24388{,}03$	$\sigma_{22} = 2257{,}41$	DOF	56

Table 4. Statistical data and t value for stretches 2 and 3, Famalicão - Guimarães

$n_2 = 24$	$n_3 = 27$	(σ_d^2)	457,79
$\bar{x}_2 = 313{,}95$	$\bar{x}_3 = 428{,}67$	(σ_d)	21,39
$\sigma_2 = 47{,}51$	$\sigma_3 = 99{,}10$	t	5,362
$\sigma_2^2 = 2257{,}41$	$\sigma_3^2 = 9820{,}92$	DOF	49

Table 5. Statistical data and t value for stretches 3 and 4, Famalicão - Guimarães

$n_3 = 27$	$n_4 = 12$	(σ_d^2)	628,06
$\bar{x}_3 = 428{,}67$	$\bar{x}_4 = 290{,}59$	(σ_d)	25,06
$\sigma_3 = 99{,}10$	$\sigma_4 = 56{,}32$	t	5,51
$\sigma_3^2 = 9820{,}92$	$\sigma_4^2 = 3171{,}82$	DOF	37

Table 6. Statistical data and t value for stretches 1 and 2, Guimarães – Famalicão

$n_1 = 67$	$n_2 = 8$	(σ_d^2)	684,33
$\bar{x}_1 = 293{,}46$	$\bar{x}_2 = 286{,}31$	(σ_d)	26,16
$\sigma_1 = 79{,}46$	$\sigma_2 = 68{,}71$	t	0,27
$\sigma_1^2 = 6313{,}66$	$\sigma_2^2 = 4720{,}80$	DOF	73

Table 7. Statistical data and t value for stretches 2 and 3, Guimarães – Famalicão

$n_2 = 8$	$n_3 = 21$	(σ_d^2)	975,78
$\bar{x}_2 = 286{,}31$	$\bar{x}_3 = 360{,}06$	(σ_d)	31,24
$\sigma_2 = 68{,}71$	$\sigma_3 = 89{,}99$	t	2,36
$\sigma_2^2 = 4720{,}80$	$\sigma_3^2 = 8099{,}26$	DOF	27

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From the data analysis it results that the deflections recorded in the Famalicão – Guimarães direction are substantially different, for any target probability, while in the Guimarães – Famalicão direction stretches 1 and 2 are not statistically significantly different.

Table 8. Limit values for the t-coefficient for various degrees of probability

DOF	Probability, p			
	0,1	0,05	0,01	0,001
20	1,72	2,09	2,85	3,85
21	1,72	2,08	2,83	3,82
22	1,72	2,07	2,82	3,79
23	1,71	2,07	2,82	3,77
24	1,71	2,06	2,80	3,75
25	1,71	2,06	2,79	3,73
26	1,71	2,06	2,78	3,71
27	1,70	2,05	2,77	3,69
28	1,70	2,05	2,76	3,67
29	1,70	2,05	2,76	3,66
30	1,70	2,04	2,75	3,65
40	1,68	2,02	2,70	3,55
60	1,67	2,00	2,66	3,46
120	1,66	1,98	2,62	3,37

As a result, in the Guimarães – Famalicão direction only two homogenous stretches have been considered. The new division is presented in Table 9.

Table 9. Homogenous road sectors – second division

Guimarães - Famalicão	
1	0→ 1+526 m
2	1+526 → 1+966 m

The readings obtained on these two stretches are again analyzed to see if they are significantly different, as shown in Table 10.

Table 10. Statistical data and t value for stretches 1 and 2, Guimarães – Famalicão

$n_1 = 75$	$n_2 = 21$	(σ_d^2)	467,66
$\bar{x}_1 = 292{,}70$	$\bar{x}_2 = 360{,}06$	(σ_d)	21,62
$\sigma_1 = 78{,}41$	$\sigma_2 = 89{,}99$	t	3,12
$\sigma_{12} = 6148{,}63$	$\sigma_{22} = 8099{,}26$	DOF	94

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The results are considerably different for these two homogenous stretches. In Figure 6, the homogenous stretches with their (approximate) kilometric positions are presented.

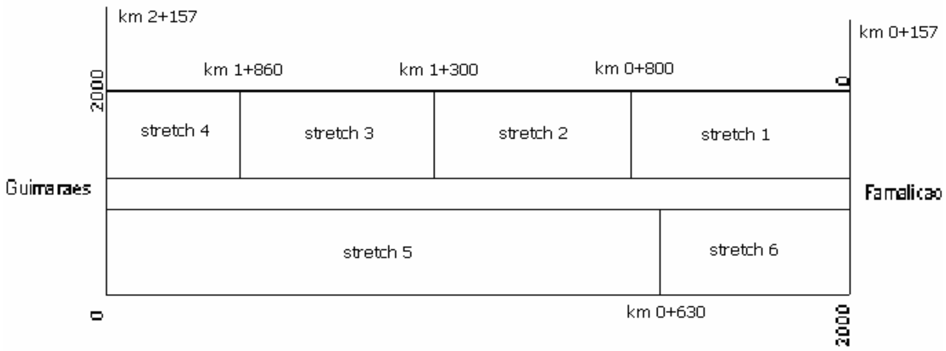

Figure 6. Homogenous sectors on the studied road

4. CONCLUSIONS

The present paper presents the work carried out by the author in the frame of "Leonardo da Vinci" Student Mobility Program, Contract RO/2004/PL93209/S, at Universidade do Minho - Center for Civil Engineering, in Portugal.

This work aimed at designing the rehabilitation of the national road "EN 206 Variant" between Carreira and Guimarães, in Portugal. A brief description and the main achievements of the work undertaken was presented. A detailed description of the procedure for division of the road into homogenous sectors recommended by COST 336 Action Final Report was also presented.

This procedure is easy to be used and grants good results which, in this case, matched the field visual observations and the results of the laboratory tests.

It also appears to be easily adaptable for creating a computer program, since most of the steps involve statistical data analysis.

Human decision intervenes when deciding what the portions of the cumulative sum graphics with constant slope are. However, this can also be included in a program application. A possible approach would be to analyze mathematically the slope variance, starting from the finest division possible, and then to move forward with joining consecutive sectors, as long as the homogeneity condition is fulfilled, until the criterion of significant difference between consecutive sectors is attained.

As recommended by Bennett, Christopher et al. [2], data transformations should be done in two steps: first, the data are transformed from source data into what it is called smallest common denominator sections. These are the smallest intervals that correspond to all data and the analysis sections. The smallest common denominator data are then amalgamated so that they can represent the conditions of the analysis section.

Such an application could be a useful tool for pavement management at both project and network levels.

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Methodology used in a recent highway construction in Portugal

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Summary

During the last years, Central and Eastern European countries aimed to increase the participation of the private sector in the construction of new highways and in the development of the transportation network. Presently, public funds are scarce to support the call for transportation infrastructures and the network development in the new EU countries, since projects and the construction of highways involve a high capital investment and an extremely long amortization. So, a possible solution for an adequate risk management of the public funds is the private-public partnerships (PPP), namely through the use of concessions (i.e. construction and exploitation of the public highways network by private entities). One of the European countries which is widely using the concession model in its Road National Strategic Plan is Portugal. This paper is based on the work carried out at the Department of Civil Engineering of the University of Minho, within the Highways group, in the frame of "Leonardo da Vinci" Student Mobility Program, Contract RO/2004/PL93209/S, and the main objective is to study the Portuguese experience on the use of the concession model, in order determinate its main advantages and disadvantages. A case study on the use of the concession model was followed for four months during the construction of some new stretches of highways located near Oporto city.

KEYWORDS: private-public partnership, transportation infrastructure, concession, concession stages.

1. INTRODUCTION

The need of improving the infrastructures network of a country, especially in the transport sector, is seen as an essential condition of a successful economic growth.

At a European scale, the High-Level Group of the Trans-European Transport Network (TEN-T) confirmed the need of reformation in the current trans-European transport network guidelines, especially after the 2004 and 2007 European Union inclusion of 12 new countries and the resultant enlargement of boundaries [1].

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One of the potential solutions for the reorganization of the European transportation network is the involvement of the private sector. Private-public partnerships (PPP) can provide and operate transport infrastructure facilities and services that were once seen as natural monopolies which should be provided and managed exclusively by the public sector [2].

Through the establishment of a partnership between the public and private sectors, concessions can be an effective means of satisfying the strategic needs of highway transportation agencies [3].

The main reasons for using a concession model range from a lack of public funding to a belief that private financing and delivery provide a higher quality [3]. Public-private partnerships can provide an important share of private capital. They essentially require a greater transparency of costs, what obliges the public authorities to have a more strict management and to clarify their long-term policy (regulation, infrastructure charging) committing themselves contractually, so as to reduce the risks [1].

2. SOME EUROPEAN PRACTICES AND THE PORTUGUESE EXPERIENCE

Governments have to manage public highways as a result of the lack of inherent incentives supported by the private sector. In some European countries, however, there is a belief that the private sector can provide higher quality services at a lower cost. In other countries, the public sector is not capable of or is not willing to make the financial investment required to complete major infrastructure projects. These are just some of the reasons for the use of concession contracts as a part of highways agencies' long-term strategic network plans [3].

Many European highways agencies are beginning to take the role of network operators rather than providers of services, thus leading to an outsourcing of production tasks through concession contracts. A concession contract is present whenever the concessionaire carries out the whole capital investment, operates the resulting service and is remunerated through service fees paid by users. Moreover, the facilities are to be handed over to the oversight public authority at the end of the contract period [3].

Table 1 clarifies the differences between concessions and other PPP. The option in column 1 of Table 1 provides the spectrum of PPP from traditional public agency management to complete privatization [4].

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Some European countries are aggressive users of the concession model, since they believe that concessions will provide a better value for each spent Euro. Concessionaires are seen as an extension of the highways agencies.

Table 1. Different types of public-private partnerships [4]

Option	**Capital Investment**	**Operation & Maintenance**	**Commercial Risk**	**Asset Ownership**	**Contract Period**
Public Agency Management	Public	Public	Public	Public	
Service Contract (Performance Contracting)	Public	Public/ Private	Public	Public	1 to 2 years
Management Contract	Public	Private	Public	Public	3 to 5 years
Concession of Existing Network	Private	Private	Private	Public	5 to 30 years
Concession of New Facility (Build, Operate, Transfer)	Private	Private	Private	Public => Private	20 to 30 years
Privatization	Private	Private	Private	Private	Indefinite

Table 2 lists the financial and political advantages of using concessions for the administration [4].

Table 2. Advantages of using concessions [4]

Financial Advantages	**Economic & Social Advantages**	**Political Advantages**
• Easing of budgetary constraints • Optimal allocation and transfer of risk to the private sector • Realistic evaluation and control of costs	• Streamlined construction schedule and reliable project implementation • Modernization of the economy and improvement of services • Access to financial markets, combined with the development of local financial markets	• A new role for the public authority • Allocation and not "abdication" • Project stability

France and Portugal are the most aggressive users of concessions in Europe. In France, concessions have been an integral part of its program to develop, operate, and maintain its main highways for more than 30 years. Portugal is aggressively employing concessions as part of its strategic plan to develop its national highway system, and about 90 percent of that system is controlled by concessionaires [3].

The primary factor leading to the Portuguese concession plan was the entry of Portugal in the EU and the need of strengthening its trading ability [5]. The Portuguese public agency for highways (*Estradas de Portugal*, EP) has made major

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changes in its form of approaching to the need of a highway network development. In 1991 Portugal's roadway network included only 431 km of concessions. In 2006 it has a total of 2700 km of built concessions, thus representing 90 percent of its national highway network. The concession model allowed Portugal to complete its strategic National Road Plan eight years earlier than scheduled [3].

Financially, the State budget could contribute towards the initial investment up to 35% and towards the economical equilibrium of the concession [3]. Moreover, the Portuguese concession contracts use two primary payment vehicles: i) real tolls, through which concessionaires finance and maintain the roadway in return for payments collected as tolls from users; ii) shadow tolls, through which the government compensates the concessionaire based on the number of vehicles which use the roadway. The system with shadow/virtual tolls (known in Portugal as SCUT) was introduced whenever a motorway was required and there was no good quality alternative, or the traffic forecast was not considered to be interesting enough as to bring sufficient competition between bidders.

In a concession strategy such as that developed by the Portuguese, appropriate risk allocation is essential. The risk-control strategy suggests that the party more capable of managing the risk supports it. For instance, the risks associated with design, construction, operation and maintenance, latent defects, and legislation are assigned to the concessionaire, while there is a shared responsibility for environmental actions, land acquisition, and force *majeure* events. Planning is the only risk which the government fully maintains [3].

Two of the most difficult risks which affect transportation projects are right-of-way acquisition and environmental approval. EP’s preference is to obtain environmental approval before launching its program or to retain the risk of failure so as to obtain approval. Many of the projects are subjected to environmental problems which result in a delayed beginning of the payments. When this occurs, the government compensates the concessionaire for additional costs, what can be very expensive.

Right-of-way acquisition cannot be totally delegated to the concessionaire because expropriation (condemnation) rights may be only exercised by the government. The first Portuguese concessions gave the government primary responsibility for acquisitions. This method has proved to be burdensome. The most recent concessions have significantly reassigned the right-of-way risk to concessionaires. Concessionaires handle negotiations and the government provides the public interest declaration. If it is contested, the matter goes to court and the government handles the case and the potential risk of delay in the court proceedings.

The loss of the owner and valuable EP’s expertise is one of the adverse impacts of the aggressive Portuguese concession program, because it has enabled the EP to downsize its engineering and administrative staff. In fact, EP must keep on developing design, construction and operation standards, and policies which will be the basis for the establishment of the scope of the concessionaires' duties. The loss of the expertise will be felt for many years, both in the lack of resources to review future concession proposals and in the administration of current contracts [3].

Moreover, there are other examples of PPP in Europe. The United Kingdom has commitments or plans for more than 15 projects to date. The Netherlands have embarked on a limited use of concessions (tunnel and rail projects), and now they are trying to contract concessions for smaller maintenance/operations woks [3].

The experience with concessions diverges among the different countries in Central Europe. In Hungary, there was a strong policy to encourage the development of highways on a privately financed basis and the government actively promoted the development of several concessions. In Poland, public-private partnerships are favoured under the management of the Motorway Agency. In the Republic of Croatia, a Toll Road Authority has been established in order to oversee the execution of the motorway network with the participation of the private sector. In the Czech, Slovakian and Slovenian Republics, there has been no development of highways on a concession basis. The highway network is being essentially developed on a public financed basis (contribution from fuel taxes etc.) [6].

There are arguments both pro and against the concession model for Eastern European countries. While the private involvement can fill important financial gaps, the institutional difficulties make this system still difficult to apply. Actually, it has been noticed that, due to the adverse institutional conditions prevailing in the transition period, high transaction costs and unrealistic demand expectations, PPP in Central and Eastern European countries have been less successful than in other countries and certainly less successful than it was initially expected [7].

Problems arisen from the implementation of PPP programs in EU countries are also connected to the poor management of pre-qualification steps, for the sorting of bidders with required financial means and expertise. Moreover, it was concluded that the management of a concession agreement should be simplified and monitoring schemes ought to be implemented in order to prevent a contractor's opportunistic or free-rider behaviour [8].

3. PRESENTATION OF THE STUDIED HIGHWAYS STRETCHES

This study refers to the stretches of new and existing highways which are being built (or reconstructed) near Oporto city by using the concession contract model. These road infrastructures are integrated in the concession *SCUT of Greater Oporto*, which congregates a group of roads and highways of the Portuguese National Road Plan, placed in Greater Metropolitan Area of Oporto. The stretches of roads under construction, presented in Figure 1, were visited during the

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scholarship period of four months and they are the main object of study of this project.

Some of the stretches of roads of the concession under construction will only be broaden through the increase of the number of lanes, because of the high occupancy of these roads and their extremely low level of service (16.3 km of IC 25, IP 4 and IC 24). However, the building of new stretches of highways is the main segment of the observed work, with a total length of 39.3 km distributed along IP 4, VRI, IC 24, IC 25 and EN 207 [9].

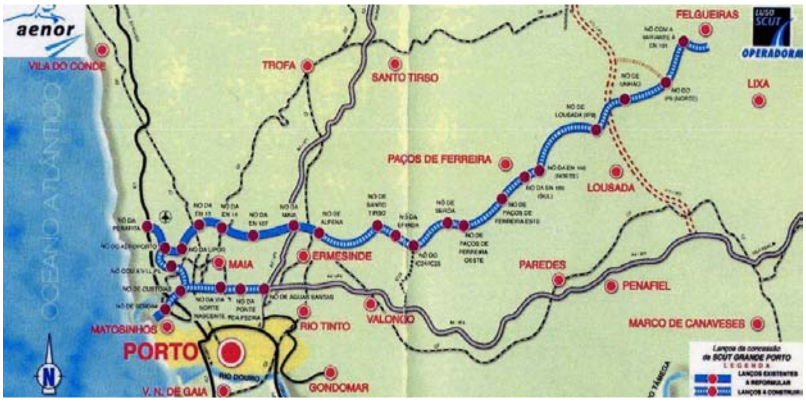

Figure 1. Stretches of road being built in the concession *SCUT of Greater Oporto*

The new roads and highways, which are being built in this concession, will work out as ring roads for the urban area of Oporto (namely, the new stretch of IP 4 and the IC 24), as local distributor roads (IP 4, IC 24 and VRI, linking IP 4 and IC 24) and as through highways which link the outlying cities to the nuclear centre of greater metropolitan area of Oporto (IC 25 and EN 207).

According to the available elements of traffic, it was considered, during the public competition phase and in the negotiation phase to attribute the concession, the implementation of a profile with four lanes in each direction, thus occupying a total platform width of 36.60 m.

The temporal macro objectives established by the concessionaire and the contractor for the design and construction of the roads are the following:

- Conclusion of the project phase : December 2004
- Conclusion of the expropriation phase: December 2004
- Construction beginning: November 2003
- Construction deadline: September 2006

The negotiation volume of the contractor with the concessionaire was fixed at 763.0 millions of euros (project, construction, traffic counting equipment, other expenses and management) for a total construction length of 55.6 km.

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4. CONCESSION METHOD: THE GREATER OPORTO CONCESSION

The roads and highways which are being built in the greater Oporto concession, as well as the headquarters and the laboratory of the contractor, were visited during the four months of this study. As an example of the Portuguese concession process, a new concession method was described (entities involved in the process, their contractual interactions and obligations).

The main entities involved in the concession process are the following:

- Owner – entity who gives the concession or the right to perform some type of business activity in its own lands or properties;
- Concessionaire – entity who has been given a concession to perform the business activity;
- Contractor – entity responsible, before the Concessionaire, for the punctual execution of the project and construction of the highway stretches;
- Sub-contractors – entities hired by the contractor, which are responsible for the conception and project of the highway stretches (designers) and for the construction of the several highway stretches of the concession.

The Portuguese Government owns the greater Oporto concession, whose agent responsible for giving road or highway concessions is the public entity *Estradas de Portugal* (EP).

The greater Oporto concession contract integrates the conception, construction, duplication and increase in the number of highway lanes, financing, exploitation and conservation, in a system with shadow or virtual tolls (SCUT without user's fee) for a period of 30 years. This concession contributes in an undeniable way to a better life quality of the people who live or work in the district of Oporto and to the economic and social development of the country. In fact, through the 56 km of highways of the greater Oporto concession, it is possible to:

- have access to an alternative network of great speed roads, which links the metropolitan area of Oporto with the Northern municipalities of that district;
- have access to the border with Spain through a fully highway connection in an hour and a half;
- have access directly to the local airport and harbour, without needing to drive through the city.

The concessionaire which has been given the greater Oporto concession in 2002 is the Lusoscut, Highways of Greater Oporto (Lusoscut GO). This is a group of several construction companies associated with some banks. The Lusoscut GO is one of the five concessionaires of the Aenor organization, which is responsible for 600 km of the Portuguese highways given in concession.

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In Portugal, the main concessionaires are Brisa Group, Aenor Group (in which Lusoscut GO is included), Highways of Atlantic Group, Euroscut Group, Lusoponte, Norscut and Scutvias.

The contractor associated with the greater Oporto concession, which is responsible for the execution of the project and construction of the highway, is the Complementary Group of Companies (ACE in Portuguese) designated as Portuscale. The ACE Portuscale is the contractor linked to the concessionaire Lusoscut GO, since these two entities were formed by the same group of associated construction companies.

Thus, the ACE Portuscale is basically the contractor branch of the Lusoscut GO concessionaire, and the construction companies associated with the ACE are the sub-contractors responsible for the construction of the several stretches of the concession's highways. The ACE designers are the entities responsible for the conception and elaboration of the project for the highways of the concession GO.

In this concession model, the owner gives the concession to the concessionaire by setting down a Concession Contract (CC) to regulate the rights and duties of the concessionaire, which can be divided into the following sub-contracts:
- Conception, design and construction of the highway;
- Financial support of the construction;
- Maintenance and exploitation of the highways of the concession.

During the period of concession (thirty years in the case of GO), the concessionaire must construct and maintain the highways at a good service level, by using their own funds. In order to be compensated for this huge investment, the concessionaire can benefit from the concession exploitation, essentially by receiving real tolls (users) or shadow tolls (SCUTS).

The Project and Construction Contract (PCC), which is enclosed in the CC, regulates the relationship between the concessionaire and the contractor. The PCC objective is to define the phases of conception, design and construction of the highways of the concession, by the contractor, in a fixed and global price system, with a specific deadline for the conclusion of the work. This contract represents an integral sequence of the concessionaire responsibilities before the owner, for what respects to the conception, project and construction of the highway.

The Project Contract (PC) legalizes the trade between the contractor and the companies which carry out the highway projects of the concession (coordinators, verifiers and designers). The PC defines the objectives and conditions required to conceive and design the projects of the highway. These are essential to begin the construction of the highways of the concession on time and to obtain a final product of good quality.

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The Sub-Contract Agreement (SCA) normalizes the relationship between the contractor and the sub-contractors responsible for the punctual construction of the highway stretches. The SCA is also a full sequence of the contractor obligations before the concessionaire and, consequently, of the concessionaire before the State, concerning the construction of the several highway stretches. Thus, based on the SCA, the sub-contractors must execute and conclude all the construction works on the several stretches of highways without delays.

Each entity involved in the concession model has specific obligations. The main responsibilities of the owner are the activities of expropriation and the actions of approval of all the studies and projects needed to close up the concession contract.

The concessionaire must exploit the concession acceptably, by maintaining the concession at a good service label and assuring a profitable outcome. Thus, it has to negotiate and assure an adequate relationship with the owner and with the financing institutions (banks), by also mediating the defence of the contractor rights before them. During the construction of the highway, the concessionaire assumes the role of owner of the construction.

The duties of the contractor are related to the supervision and coordination of the several entities involved in the construction process and the certification of reliability of the projects, procedures and inspections made by the designers, verifiers and other sub-contractors. The contractor must also assist the sub-contractors when dealing with external qualified entities and act as an intermediary body in the safeguard of their legal rights before the concessionaire.

The main obligation of the sub-contractors is to construct the highway stretches of the concession as established in the sub-contract agreement, which is in conformity with the project and before the deadline established in the SCA, by following strictly the safety and environmental rules defined in the project.

The main phases of the concession are the project, the construction and the exploitation of the highways of the concession. These three phases are included in the concession contract and comprise:

- the Construction Programme, which encloses the studies and projects, the expropriations and the construction of the highways of the concession;
- the Programme of Major Maintenance and Enlargements, during the exploitation phase of the concession, which also includes the respective project.

This paper is essentially focused on the concession phases observed during the several visits to the contractor (Portuscale). The contractor activity comprises the project and construction phases, which are included in the PCC and are planned out in the construction programme (also called Plan of Studies and Projects, PSP). The PSP specifies the period of the various phases of the project and construction. As these are the main phases observed *in situ*, they will be described next.

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5. PROJECT PHASE OF THE CONCESSION

The project of a highway is formed by a specific number of volumes for each type of work. Every volume has a descriptive memoir and specific drawings to detail the work. A report on the environmental conformity of the execution project must be also annexed to the project (in Portuguese, *Relatório de Conformidade Ambiental do Projecto de Execução* – RECAPE). The main contents of a highway project are:

- Vol. 1 – Synthesis;
- Vol. 2 – Setting out and surveying support;
- Vol. 3 – Geological and geotechnical studies;
- Vol. 4 – General alignment/layout;
- Vol. 5 – Junctions;
- Vol. 6 – Re-establishments, parallel roads and other passageways;
- Vol. 7 – Drainage;
- Vol. 8 – Paving works;
- Vol. 9 – Landscape integration;
- Vol. 10 – Safety equipments;
- Vol. 11 – Traffic signing;
- Vol. 12 – Equipment to count and classify the traffic and closed-circuit TV;
- Vol. 13 – Telecommunications;
- Vol. 14 – Lighting;
- Vol. 15 – Fences;
- Vol. 16 – Affected services;
- Vol. 17 – Usual engineering structures (e.g. overpasses and underpasses);
- Vol. 18 – Special engineering structures (e.g. viaducts and long bridges);
- Vol. 19 – Ancillary projects (e.g. retaining walls);
- Vol. 20 – Expropriations;
- Vol. 21 – Measures to reduce the highway impact (e.g. noise barriers);
- Vol. 22 – Operation and maintenance centre;
- Vol. 23 – Service facilities, street furniture and picnic areas;
- Vol. 24 – Tunnels.

The relationships between the main entities involved in the project phase of the concession are presented in Figure 2.

The objective of this organization is to assure the punctual execution of the studies and projects of the highway, with the required quality, and to respect the financial plan defined in the concession contract.

The beginning of the construction phase depends on the project phase which includes fundamentally the following three aspects:

- Delivery and approval of the expropriations project, in order to begin the process of expropriations, which is a responsibility of the owner and which is carried out during a further period of 6 months;

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- Delivery and approval of the final and complete project of reestablishment of the affected services, in order to contact promptly the several external entities (by controlling the affected services) and thus allowing the urgent removal of these services at the beginning of the construction works;
- Delivery and approval of the execution project for the highway or, at least, of the drawings needed to start the first construction works.

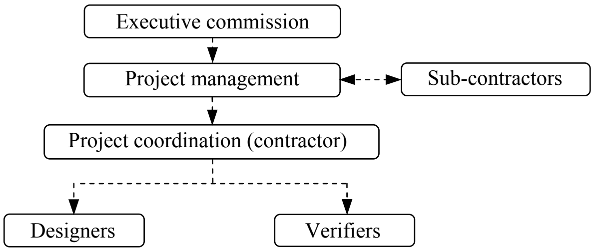

Figure 2. Organization of the main entities involved in the project phase

The expropriation project is organized by the contractor, namely by evaluating and proposing a unitary value for the expropriated lands. However, the owner should approve that project and start the expropriations process by acquiring those lands.

The expropriation phase must be finished before a 6 months period. The expropriated lands can be taken over by the Portuguese state (EP), without any legal impediment, since the highway is a construction of national interest. Nonetheless, lands with inhabited houses can only be expropriated after the residents leave their house, and EP must compensate the inhabitants for their land and house. The value of the compensation can be determined by mutual agreement, but sometimes it is a legal verdict given by the judge in the court of law, thus extending greatly the period of expropriations (more than 1 year). The tenants who live in these houses must also be compensated by the state (with money or a new house).

The construction of the highway usually begins before the end of the expropriation phase. The lands not yet expropriated and their accesses will obstruct the normal advance of the works *in situ*, thus delaying and probably compromising the contractual deadline to build the highway. In this case, the owner must compensate the contractor and the concessionaire (e.g. by extending the concession period).

Regarding the services affected by the new highway, they are mainly gas pipes, water pipes, sewer pipes, telecommunications and electrical cables and poles. The reinstallations of the affected services are usually projected and carried out by the designers and sub-contractor companies working with the contractor. Nevertheless, the telecommunications and the electricity reestablishments are carried out by the external companies responsible for those services, and the contractor can only

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negotiate the value to be paid for those works. Basically, the contractor has the role of mediator with the several external entities, by reaching an agreement as quickly as possible (to avoid delays during the construction), by negotiating the cost of the reinstallation works and by solving the problems of the sub-contractors. The main phases for the reinstallation of the affected services are the following:

- 1st phase – After finishing the preliminary study and before starting the geometrical layout – Looking for information about the affected services in the external entities;
- 2nd phase – At the beginning of the carrying out of the execution project – Emission of the project with the record of the affected services;
- 3rd phase – Until the end of the production of the execution project – Emission of the project with the solutions for the repositioning of services.

6. CONSTRUCTION PHASE OF THE CONCESSION

The organization of the construction process is coordinated by the contractor administration, whose main objective is to guarantee the execution of the construction works before the deadline, within the quality and budget limits [10].

The contractor activity during the construction phases fundamentally comprises the achievement of the following objectives:

- To guarantee the accomplishment of the contractual deadline through an adequate Construction Plan by following the work evolution;
- To guarantee a final product with the requested quality through inspections and tests and through adequate solutions to eventual non-conformity situations;
- To control and to process all the paperwork (including the invoices to the sub-contractors and to the concessionaire);
- To guarantee health and safety conditions at all workplaces and job sites;
- To follow and deal with any environmental and archaeological occurrence on site without affecting the normal work evolution;
- To guarantee the correct management of the insurance policy.

Concerning the planning of the highway construction, any sub-contractor must produce and submit an Initial Work Plan to the contractor until the 30th day after reception of the detailed project (limited up to 10 days before the consignation). Once approved, this Work Plan becomes the reference document for all construction works. The content of this initial work plan can be listed as follows:

- Explanatory Report;
- Work Programme with physical and financial chronological diagrams;
- Charts of equipment and labour force requirements;
- Work yard Project and accesses, circulation and road sign plans;
- Organisational Diagram;

- Expropriations Chronological Diagram.

The contractor has two procedures to follow up the construction work, namely the Work Programme Update (performed in a monthly basis) and Work Programme Revisions (performed every three months).

Concerning the Work Programme Updates, any sub-contractor must schedule and submit the initial work programme to the contractor until the 10^{th} day of each month, with an analysis that justifies eventual work delays and the respective measures to recover them. These updates must always show the initial work plan or the last approved revision of the plan, registered as "Baseline", by underlining the essential sequence of procedures.

Regarding the Work Programme Revisions, they must be carried out every three months or every time the contractor considers it is necessary. The sub-contractors must submit the Reviewed Work Programme within 30 days after it has been required by the contractor. Those Revisions must explain the work delays and the respective measures and deadlines to recover the time lost, including the necessary equipment and labour force reinforcements to respect the work deadline established in the initial contract. The essential sequence of procedures must be underlined.

To allow the weekly planning of the contractor activities and the appropriate monitoring of works, the sub-contractors are requested to submit a fortnight work programme to the contractor, containing information about the current or new workplaces within the analysed period, the stop points (for Quality Control and Health and Safety Assessment) and all the necessary operations, regarding the Affected Services.

The following Work Programme must be submitted fortnightly, by the Sub-contractor to contractor, until the penultimate working day of the week, indicating the active workplaces, in order to allow the coordination of the activity of the several work agents.

In order to avoid eventual delays in the beginning of works, it is necessary to guarantee the exchange of several documents among the different parts involved in the construction (concessionaire, contractor and Sub-contractors). At the consignment date, the Contractor has to provide to the Sub-contractors the documents listed below (independently from previous deliveries of the same documents in other phases of the process):

- Approved detailed project
- Models for the emission of measuring reports, listing the articles and unitary costs settled with the Sub-contractors
- Sub-contractor’s quality manual
- The verification of the supporting polygonal (road line)
- The verification of the expropriations polygonal (area)

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- Health and Safety Plan
- CD with the Work Quality Manual, sub-contract agreement and contract of project and construction
- Application of laboratorial management – Highways

The preparation, compilation and approval of the documents presented above imply the preparation of other documents, which the Sub-contractor must deliver to the Contractor, such as those presented in Table 3.

Table 3. Documents submitted by the Sub-Contractor to the Contractor

Documents	**Delivery time limit relatively to the beginning of works**
Surrounding conditions	until 45 days before
Preliminary work programme	until 45 days before
Preliminary chart of labour force	until 45 days before
Preliminary chart of equipment needs	until 45 days before
Information concerning preliminary communication	until 12 days before
Crises management – Emergency plan	until 10 days before
Safety management – Organisational Diagram	until 10 days before
Special risks Work Plan	until 10 days before
Project of the workyard and plans of accesses, circulation and sign placing	until 10 days before
Sub-contractor quality plan	until 30 days before
Working procedures	until 60 days before
Definitive Work Plan	until 10 days before

The documents to be delivered by the Concessionaire to the Contractor, and vice-versa, are presented respectively in Tables 4 and 5 (some documents do not have an explicit deadline).

There are some fundamental definitions related to inspection, tests and non-conformity solution which must be understood, namely the following:

- Inspections and Tests Plan (or in Portuguese, *Plano de Inspecções e Ensaios, PIE*) – a document which contains a compilation of reference specifications, which demonstrate the conformity of an activity with the work quality manual, the detailed project of each highway stretch and the working procedures of the sub-contractor;
- Document analysis – an evaluation of the previous documents, necessary to begin an activity, by verifying the conformity of the materials, equipments, constructive methods or the work plan proposed by the sub-contractor;
- Stop point – situation of the work, in which the sub-contractor needs a special authorization from the Contractor to begin or continue the activity;

- Nonconformity (NC) – a product which does not conform to the specifications; serious or imminent danger situations;
- Anomaly – a failure which occurs during the works. If not corrected, it will lead to a NC;
- Correcting action (or in Portuguese, *Acção Correctiva, AC*) – an action which eliminates the causes of nonconformity, anomaly or another unwanted situation in order to avoid its repetition;
- Derogation (DRG) – a written authorization to use or deliver a product which does not conform to the specifications.

Table 4. Documents submitted by the Concessionaire to the Contractor

Documents	**Time limit**
Document with specifications approved by EP	Undefined
Detailed project approved by EP	Undefined

Table 5. Documents submitted by the Contractor to the Concessionaire

Documents	**Time limit**
Sub-contractor quality Manual	Undefined
Inspection and test plans	Undefined
Sub-contractor quality plan	Undefined
Explanatory report	Undefined
Definitive working programme	28 days after consignment

Tests and inspections must be carried out by the sub-contractor, according to the traditional control methodology and assuring the total accomplishment of the PIE. The Contractor has to verify if they are effectively and correctly accomplished. The PIE defines the inspections and tests to be necessarily performed by the sub-contractor and those to be performed by the Contractor. It regulates the inspection of the Contractor and assigns the persons responsible for every inspection action. The PIE must also present how to assess (and who assesses) the qualitative service of the sub-contractor, by showing explicit rules to accomplish the referred inspection.

The Contractor will evaluate the sub-contractor's system of quality assurance and will execute its own inspections and tests on a sampling basis (10 to 20%), which is considered to be representative of every activity, in order to validate the sub-contractor's tests and inspections. In the case of a stop point, the Contractor intervention will always consist in a previous audit of the sub-contractor (by obtaining from the sub-contractor previous copies of the inspections carried out). Moreover, the Contractor can carry out the same verifications to confirm those made by the sub-contractor.

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Another phase of the construction process is the invoicing process, in which the Concessionaire, the Contractor and Sub-contractors intervene. The methodology adopted in the process of invoicing the works carried out on a monthly basis is schematically illustrated in Figure 3.

The Contractor has the aim of establishing all the activities that should be adopted in the planning and implementation of accident prevention and safety at the worksite, namely by applying the Health and Safety Plan which results in higher levels of health, safety and comfort.

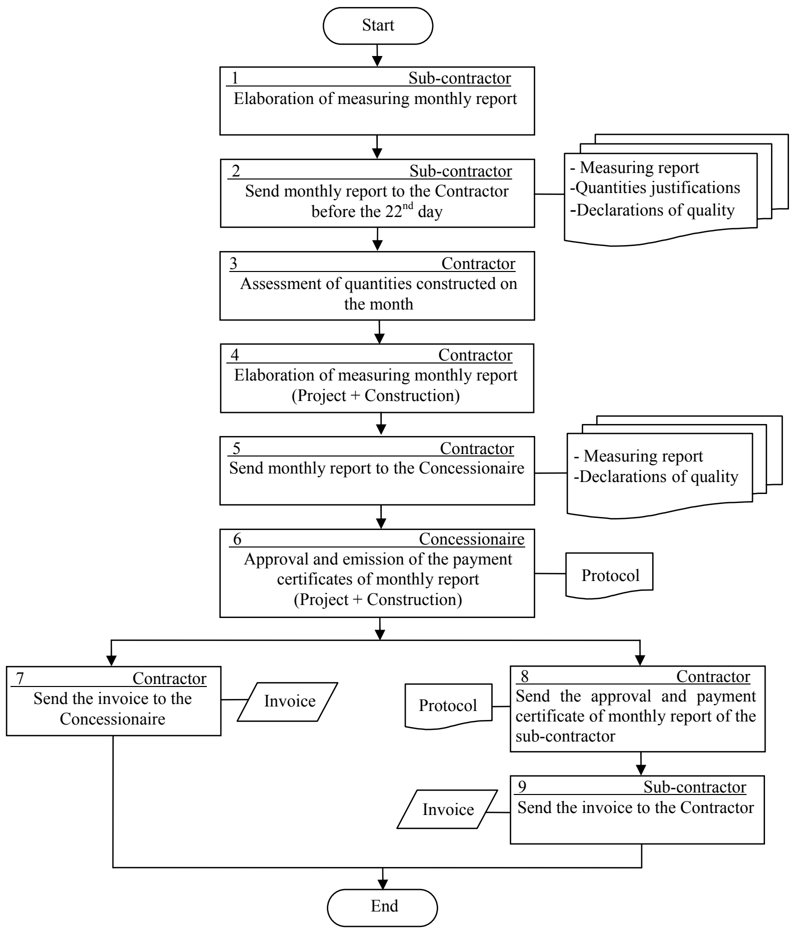

Figure 3. Methodology used in the invoicing process

This principle is applied to:

- All project and construction works of the highway sections and roads associated with the concession;

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- All the areas considered to be workyards (places which support the execution of works);
- All the areas near the workyards and worksites (in order to control the risks of human accidents).

All actions and responsibilities of the Contractor and other agents involved in the construction relative to safety are described in a specific document (*Manual Próprio* in Portuguese) by DPS (Department of Prevention and Safety).

7. CONCLUSIONS

In this paper, a new option for new EU governments (such as Romania) without sufficient monetary and human resources to construct quickly their network of highways, which is a crucial step for the economic development of these countries, was presented. The solution is the integration of private companies in the funding, project, construction and exploitation of new highways to be build, through the use of a public-private partnership known as the concession model.

The use of the concession model in the Eastern European countries, such as Romania, can be helpful (e.g. the private involvement can fill important financial gaps), but have some negative aspects (e.g. the institutional difficulties make this system still complicated to be applied). In fact, it has been noticed that the concession model in Central and Eastern European countries have been less successful than it was initially expected.

For a successful introduction of PPP, like the concession model, politicians and the general public must be confident and involved in the PPP system. Generally, the public is not informed and well prepared to accept new techniques involving private financing of public facilities. Essentially, it is difficult to understand that the development of the highways network has a great influence in the economic growth of the countries. Users will benefit directly from the tools they are paying in the newly developed highways with higher level of service.

Some of the problems in the implementation of the concession model are the shift from strategic to short-term objectives and the possibility of obtaining personal profit through the bidding process. Indeed, many opportunities for corruption exist in these transactions, and recognizing such possibilities is important when designing a successful concession program without those problems.

Nonetheless, given the substantial institutional progress in the Eastern European countries over the last years, in particular in the context of the EU access, a more fertile ground was developed for the use of PPP in the future, based on professionalism, transparency and responsibility. Now, it is time to show that the new EU countries, such as Romania, are able to make use of improved institutional capabilities to put in place efficient PPP projects.

In this context, it is important to observe the experience from other EU countries, like Portugal, which have been using the concession model with great success in the development of their national highways network during the last decades. In the frame a "Leonardo da Vinci" scholarship it was possible to observe an example of the Portuguese concession model (the Greater Oporto concession) in the phase of construction of the highway. The main advantages and problems experienced during several visits to the contractor of this concession were presented in this paper, in order to explain how a successful concession program can be implemented in the new EU countries.

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Repair and strengthening techniques for masonry arch bridges

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Summary

The paper presents a summary of the work undertaken by the first author in the frame of "Leonardo da Vinci" Mobility, Contract RO/2004/PL93209/S, at Universidade do Minho - Center for Civil Engineering, under the coordination of Paulo B. Lourenço, Professor of Civil Engineering, tutored by Daniel Oliveira (assistant Professor of Civil Engineering) and supervised by Irina Lungu (Associate Professor at "Gh. Asachi" Technical University, Iasi).

The most common techniques used in the process of strengthening damaged masonry arch bridges are here described, being revised the typical defects and damage for which they are applied, as well as economical issues and operation difficulties.

KEYWORDS: arch bridges, rehabilitation, strengthening techniques, environment pollution.

1. INTRODUCTION

Virtually, most of the defects observed in arch bridges can be repaired. Practicalities regarding the execution of various repair techniques have been fairly widely documented, although the philosophy behind the application of some of the techniques has sometimes been somewhat dubious, perhaps resulting from a fundamental lack of understanding of the structure under consideration.

Nowadays, a great variety of intervention techniques are available, from which it should be distinguished the following groups:

- Traditional techniques: they use materials and construction techniques similar to those used during the construction of the structure;
- Modern or innovative techniques: they try to adapt more efficient solutions than the traditional ones through the use of modern materials and equipments;

The choice among traditional or innovative solutions is controversial, but if with traditional techniques it is possible to obtain satisfactory solutions from the structural, economic and constructive points of view, its use should be preferred,

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not only for aesthetic and cultural reasons, but also for compatibility reasons between the new elements and the original ones. When dealing with cultural heritage bridges, the choice between "traditional" and "innovative" techniques should be determined on a case-by-case basis with preference given to those that are least invasive and most compatible with heritage values, but consistent with the need for safety and durability.

In some cases, it is not easy to repair the structural damage with the exclusive resource to traditional solutions, because original materials are no longer available, as mortars, because qualified labour doesn't exist (craftsmen) for this type of constructive techniques, or even for economical reasons. The most frequent reason to go through modern techniques is related with the need of significant increases of load bearing capacity, that are only gotten with much more efficient materials than the original ones. However, whenever possible the "interventions in masonry should be made with masonry".

Before the decision for the use of any repairing techniques or reinforcement is made, it is quite necessary to establish and to understand the causes of the found damage. On the other hand, the effect of the intervention on the behavior of the structure should be carefully evaluated, by means of in situ tests or numerical modeling.

2. IDENTIFICATION OF DEFECTS

The identification of many of the defects affecting masonry bridges (spandrel wall bulging, bowing or detachment, gross abutment movement) is straightforward, being sufficient an accurate visual inspection.

Abutment movements can be identified by the presence of a crack in the region of the crown, or by settlement of the parapet walls. Spandrel wall detachment can be identified by the presence of continuous longitudinal cracks in the arch barrel beneath the internal faces of the walls. Unfortunately, some other defects may be less evident. In these cases, either partial dismantling of the structure, coring through sections of the structure or the use of NDT (non-destructive-testing) techniques will be required. These methods would prove impractical or expensive for the majority of bridges requiring assessment, but useful for assessing a small numbers of important structures, or a sample of representative structures.

3. STRENGTHENING TECHNIQUES

3.1 Pressure pointing and grouting

Pressure pointing and grouting is an economical strengthening technique, usually involving little traffic disruption. Grouting of the contained ground above and behind an arch can be a useful measure: with suitable receptive grounds (not high in clay or silt) and in the absence of complications such as drainage systems, the method is very effective and very economical. Furthermore, it increases the assessment factor to 0.9 and improves the arch ring condition factor by filling cracks and voids in the extrados. Grout quantities can be hard to predict and considerable variation is therefore to be expected.

Figure 1 presents the operation of repointing the joints of a masonry bridge.

Figure 1. Repointing of the joints

3.2 Tie bars

Tie bars, anchored as presented in Figure 2, are used to restrain further outward movement of spandrel walls. They consist of a bar passing through the full width of the bridge, with pattress plates at each end, generally secured by a nut and washer, to provide the restraint to the wall. If the arch ring requires strengthening at the same time a more common solution is to use a concrete saddle which will also relieve the spandrel wall of outward forces.

One of the advantages of using tie bars is that they can be inserted with little or no disruption to overtraffic. However, their effectiveness has never been scientifically proved and many engineers are worried that sections of the spandrel wall may fracture around the pattress plates or spreader beams, the walls then becoming potentially unstable. There is no scientific guidance as to suitable spacing the tie bars in a given structure.

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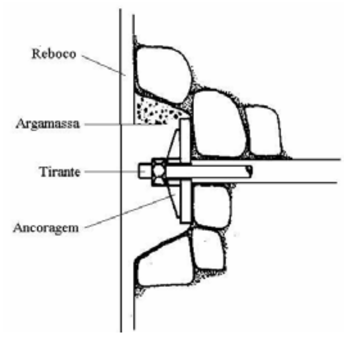
In one of the cases studied, it appeared to have been further movement of a spandrel wall since installation of the tie bars. Rusting of the exposed parts, in one severe case, was also found. The use of stainless steel bars could be considered, or even the application of cathodic protection.



a) Anchorage system

b) Anchorage plates

Figure 2.

3.3 Rebuilding bulging spandrel/wing walls

With sufficient road width or the acceptability of a road closure and with minor services present, the simple solution is to excavate behind the wall and rebuilt it conventionally. To back the wall with mass concrete is a possibility, but to do so creates a deep, stiff beam edge to the arch, inconsistent in structural action with that of the arch.

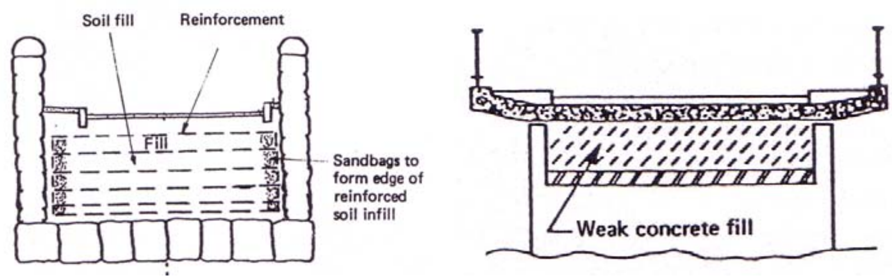


Figure 3. Strengthening of the fill material used for reducing the pressure on the spandrel walls

A more harmonious structural action results from incorporation of a reinforced earth system to support the fill, as presented in Figure 3. This prevents excessive pressure developing against the spandrel wall and the space between reinforced earth and back of wall is filled with single-size drainage material.

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3.4 Saddling

A particularly common repair technique which has been used in the case of a wide variety of arch bridges exhibiting almost any sign of distress is that of saddling (Figure 4). The technique is used in response to the observation of virtually any kind of cracks.

The merits are that with a rough existing extrados, composite structural thicknesses is increased, cracking is retained, historical widenings can be integrated, the saddle can carry a sprayed (ideally polyurethane) waterproofing membrane.

Drawbacks are that the arch is too narrow to allow single line traffic to pass while the arch is treated, due to the deep excavation necessary. Occasionally, historically widened arches may retain the old original spandrel at low level: this can be used again to facilitate ¨half and half˙ strengthening.

Saddles are typically 150-200mm thick, of relatively weak concrete and can, if judged necessary, be articulated to harmonise with the arch ring’s structural action, either by bands of transverse brickwork or an inert transverse Debonding lamina.

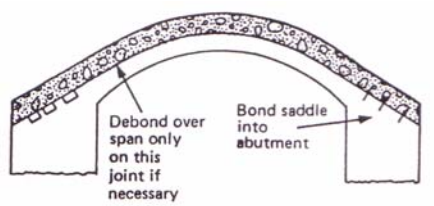

Figure 4. Saddling the extrados of the arch with a layer of concrete

Inclusion of fibers in the concrete has a merit: polypropylene fibers confer resistance to surface shrinkage cracking. Stainless steel fibers confer considerable strength and structural ability to unit arches and to bind cracks.

Saddling clearly changes the fundamental nature of the bridge and as such may often cause more problems than were originally present (e.g. the lack of stress in the original arch after saddling could give rise to the hazard of falling masonry blocks, additionally the ability of the arch to freely adjust to a changing environment is removed).

3.5 Invert slabs

An invert slab is a slab of concrete placed between the abutment walls or piers with its top surface at or below river bed level (older versions may be built of masonry). It helps to prevent scour. If incorrectly installed however, there is a risk of scour beneath the slab, particularly at its downstream end.

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3.6 Stitching longitudinal cracks

This system is applicable where more extensive dismantling or saddling is very disruptive to traffic or economically impossible.

Typically, alternate voussoir stones are cored laterally (30mm diameter) and the cores retained. A 30 mm hole is drilled normal to the spandrel and at mid-depth of the arch ring, to a length some 750mm beyond the crack to be tied. A practical maximum drilling length is about 12m. Installation of a CINTEC-type hollow stainless steel bar, with enclosing sock, takes place and the grout injected down the bar fills the sock, expanding it to key into all recesses. The CINTEC anchorage system is presented in Figure 5.

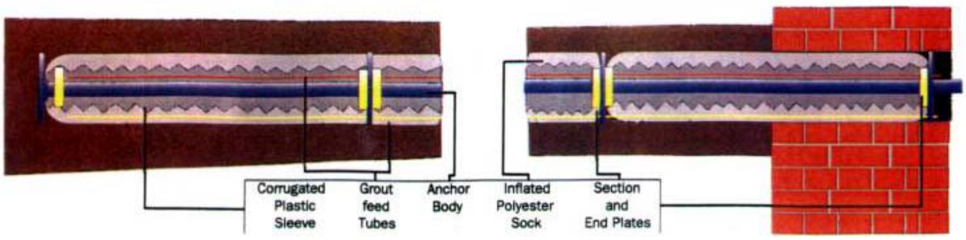

Figure 5. CINTEC anchorage system

The cracks are then pressured pointed and the ends of the stone cores reinserted to plug the holes at the face.

3.7 Guniting of Soffit

Guniting of soffit is a widely adopted technique, being non-disruptive to carried traffic and relatively economical. There are two principal drawbacks: firstly, the structure may be a cultural heritage construction, in which case the treatment would be visually unacceptable. Secondly, and of more significance for the future durability, is the failure of the method to address the most common cause of arch defects, water ingress from above. With time, this will detach the gunite skin from the arch barrel.

3.8 Overslabbing

At its simplest, overslabbing (Figure 6) consists merely of providing a load spreading slab to reduce local load intensity. It is of benefit to the barrel in that it allows the option of high-level waterproofing and it reduces lateral pressure on spandrel walls.

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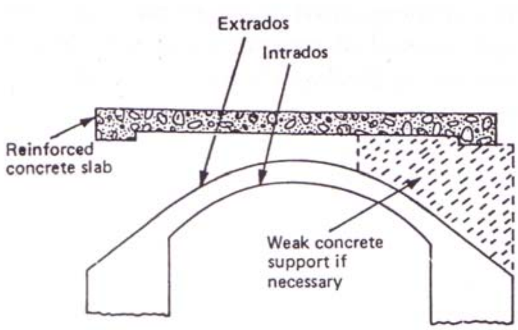

Figure 6. Overslabbing

3.9 Underpinning

Underpinning involves excavating material from beneath the foundations and replacing with mass concrete. A sequence of work is followed to ensure that the stability of the existing. Structure is not compromised. The work is labour intensive. The cases studied appeared to have been successful.

3.10 Replacement of edge voussoirs

Edge ring voussoirs are particularly prone to decomposition, due to their exposed position and the perpetual tendency for lateral load on spandrels to cause the face of the ring to detach.

To replace edge stones, it is usually necessary to provide a band of soffit shuttering to support the whole ring. While it is possible to remove stones individually without support, considerable awkward cutting is finally necessary to achieve a good soffit profile and displacement of the masonry above can occur.

3.11 Partial reconstruction

When arch ring damage is extensive, the only real resource is to rebuilt to a major extent. Construction is traditional in that it is necessary to build off centering, although several variations of constructional form have been adopted.

These are essentially mass concrete rings with articulating bands. Articulation can be achieved, at springing and quarter span points, by hard plastic formers, by bands of lime-mortar joined masonry or by open-laid bands of brickwork.

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4. MAINTENANCE

Routine maintenance consists of:

1) keeping the road surface in a good condition to maintain the waterproofing and to minimize dynamic loading from traffic due to potholes etc.
2) removing vegetation growing on the structure
3) repairing small areas of deteriorated mortar.

These three areas of maintenance involve modest expense compared with that which may result from neglect.

5. CONCLUSIONS

The problems arisen by bridge strengthening are very complex because existing bridges differ in structural materials, in construction periods, type and condition rating. The most frequent bridge building materials have been stone, wood, reinforced concrete and steel.

The causes of defects are material aging, environment pollution, poor maintenance, wrong repair works, and changes of live loads because design loads did not include the current ones due to the different type of traffic.

Any repair has to take into account not only defects and damage identified, but also the main features of the bridge, the intervention costs and operation difficulties.

Also, there are many stone bridges that are centuries-old and many of them are historical buildings, representing an important cultural inheritance. Therefore, their preservation is important, and the techniques used for their strengthening must be carefully chosen, to respond to both functional and structural demands and preserve their original characteristics.

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Numerical analysis of historical constructions

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Summary

All European countries are rich of monumental buildings and there is a considerable amount of existing residential masonry buildings in rural areas. It is frequent that old historical masonry structures present significant level of damage and must be repaired. In fact, conservation, rehabilitation and strengthening of the built heritage and protection of human lives are clear demands of modern societies [1]. This requires the identification of deficiencies and of damage of existing structures and appropriate intervention techniques. Numerical modelling of masonry structures is an important tool for evaluation of damage extent and a basis for a decision about the design of appropriate remedial measures.

The paper addresses the scope of numerical analysis of ancient masonry structures. The numerical modelling intends to assess the stability conditions of the salient elements of the façade of the Cathedral of Porto dated from the middle of 12th century. Although no important visible cracking is present, the deep decay of the granite stone and corrosion of steel bars that connect the stone pieces can induce stability problems of the local salient elements of the façade. The numerical modelling is performed with the help of a commercial finite element software program DIANA.

KEYWORDS: historical constructions, finite element analysis, smeared cracking model, load-displacement diagram, safety factor

1. INTRODUCTION

The Cathedral of Porto, as it is found today, is hybrid and frozen in a perpetual past, where re-combinations or architectural redesign seem hardly acceptable in the near future. In fact, the cathedral is a result of the previous restoration that represents today the real self of the cathedral, transformed into a monument. This is forged identity that cannot be considered a minor representation of the strength of a specific cultural period [1].

The restoration works carried out on the Cathedral during 20th century were mostly concentrated in the towers and in the roofs and façades of west and south wings.

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The central façade is composed of several salient elements, namely a central balcony and two lateral pinnacles, see Figure 1. The advanced degradation state of the granite of the columns below the referred structures led to the positioning of an auxiliary structure in order to avoid injury of people. This problem represented the main reason for the need of assessment the stability conditions of the main salient structural elements of the façade: the balcony and the pinnacles.

Figure 1. View of the two salient elements from the central part of the main façade of the Cathedral

2. DESCRIPTION AND VISUAL INSPECTION OF THE SALIENT ELEMENTS

The balcony substructure consists of an almost semi-circular slab, made of granite ashlars that seem to be bounded together with iron dowels. The slab is stiffened by three ribs of the same granite, a central one and two marginal. The loads that are considered to be applied in the balcony include the own weight and the massive granite balustrade at the top of seven granite columns that also seem to integrate steel dowels, see Figure 2.

Pinnacles are massive granite elements, but it is difficult to see whether the ashlars are connected with steel dowels or not. The main structural element of these assemblies is the shell, which takes over the loads from the two pinnacles and transmit them to the columns, see Figure 3.

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Figure 2. Details of the connection of the stones in the balcony's balustrade and between the ashlars of the slab

Even if no important cracking is observed on these elements, granite presents signs of weathering, which may result from combined action of cyclic variations of the moisture content, salts (considering also that the cathedral is situated close to the Douro river) and pollution.

As is shown in Figure 3, besides the grey spots on the surface of granite, superficial breaking off is also visible, which can be associated to some physical and chemical actions, leading to high porosity and consequently to the lowering of the mechanical parameters.

It is probable that the pinnacles' stone ashlars were connected to the battlement by mortar joints and iron dowels. However, a detachment of the pinnacles form the battlement is observed and can be attributed to deterioration of the mortar joints and corrosion of the steel bars. This can affect the deformation of the structure occurs and, thus, must necessarily be considered on the geometric model of the structure in terms of proper boundary conditions, see Figure 4.

Where structures have been changed, differences on the behaviour of the different periods of masonry may produce signs of distress. For investigating the degradations that occur in the salient elements of the façade and their connections, a GPR inspection was performed and is described in the next section.

3. GPR INSPECTION OF THE SALIENT ELEMENTS OF THE FAÇADE

The main objective of the geotechnical radar inspection (GPR) was the detection of the steel connectors that connect the various stone elements that constitute the church façade. The inspection was performed in the connection between the central balcony and the lateral battlements.

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Figure 3. Deterioration of the granite – details in the slab, pinnacles and columns

Figure 4. Detail of the detaching of pinnacles' structure from the battlement and view of the iron dowels on the back of the elements connecting the granite ashlars

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Generally, relative short radar profiles were obtained due to the small dimensions of the testing places. The balcony is composed of overlapped stone ashlars, fixed to the wall and supported, in the inferior part by stone ribs that seem to be connected with metallic dowels. The metallic connectors that may be seen in the surface of the balcony remained uncovered due to the corrosion leading to the disintegration of the stone covering layer. The objective was also to detect the steel connectors (visible or not) that existed in the elements. The lateral battlements were also investigated; see Figure 5 and Figure 6.

(a) (b)

Figure 5. Central balcony: (a) inferior ribs of the slab, (b) steel connectors in the balcony's slab

Figure 6. Left battlement of the facade

On the top surface of the balcony various radar longitudinal (parallel to the façade) and transversal profiles (perpendicular to the façade) were performed. It was observed that the ashlars are not connected to the wall by steel connectors. The unique steel connectors detected were the ones visible on the balcony's surface.

The most relevant results of the inspection may be seen in the Figure 7, where the radargram of the longitudinal profile of the slab is shown. In this profile, the ribs that support the base of the balcony are visible as reference hyperboles. The

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average propagation velocity was about 11.5 cm/ns, which is the usual value for the granite stone. No signs of cracking or any other deterioration were detected.

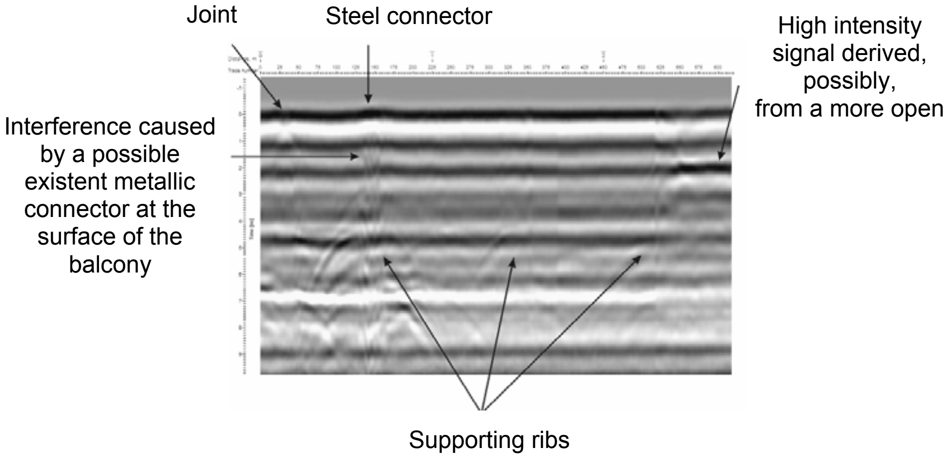

Figure 7. Longitudinal radar profile at 15 cm from the facade’s wall

On the bottom surface of the balcony some transversal profiles over and between the ribs were performed. The results obtained in the profiles over the ribs are shown in Figures 8, 9. From both profiles, it is possible to observe the thickness of the ashlars and the position of the steel connectors on the opposite surface (that corresponds to the top surface of the balcony). It is also possible to verify the presence of steel connectors in a depth of around 3 to 6 cm from the top.

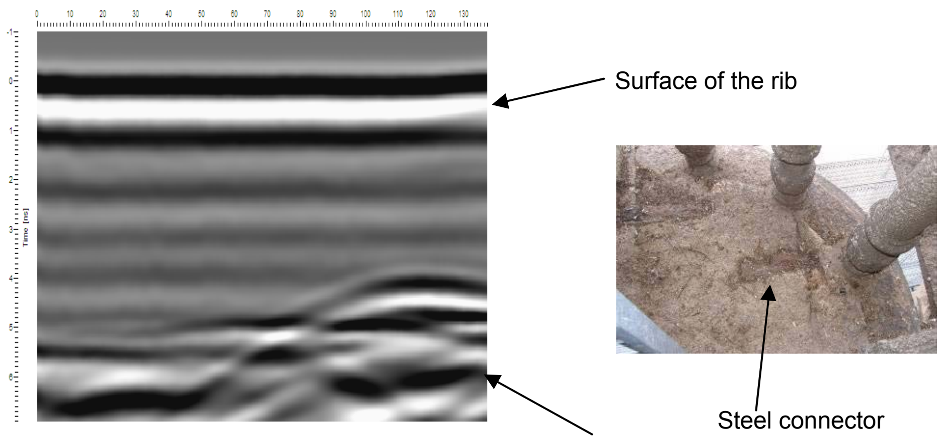

Figure 8. Radar profile on the left supporting rib

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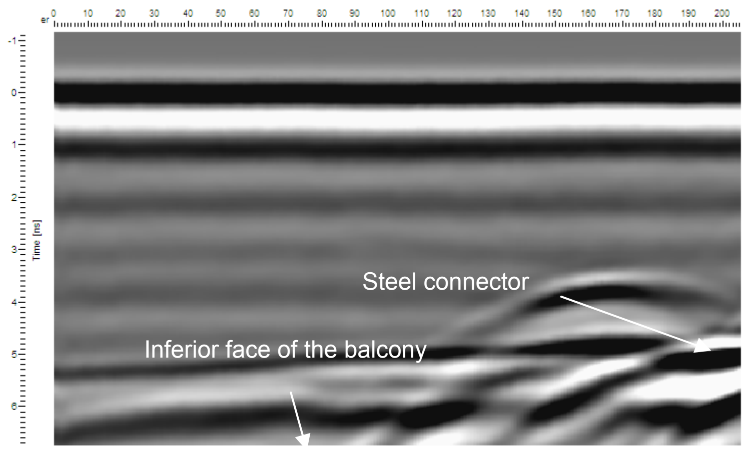

Figure 9. Radar profile of the middle supporting rib

In the left side battlement four tests (two horizontal and other two vertical) were performed. The existence of two steel salient elements that connect two granite independent ashlars made impossible the radar inspection in that respective influence area. Even so, neither deficiency nor steel connectors were detected in this zone, see Figures 10, 11.

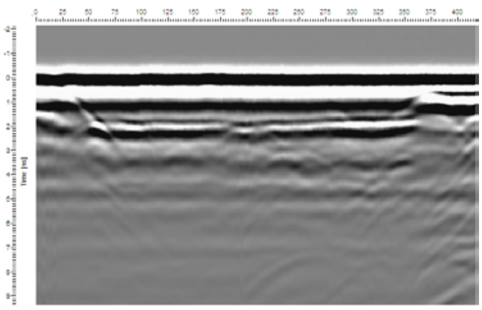

Figure 10. Horizontal radar profile

4. STRUCTURAL ANALYSIS OF THE TWO SALIENT ELEMENTS OF THE MAIN FAÇADE OF THE CATHEDRAL OF PORTO

In general, historic heritage buildings have complex geometrical shape, which is also the case of the two elements of the façade under analysis: the balcony and the pinnacles. As both structures are clearly three-dimensional and for better simulating their mechanical behaviour, solid elements were adopted for the

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calculations. Thus, finite quadrilateral (type CHx60, 20 nodes) and triangular (CTP45, 15 node) volume elements were used to create the finite element mesh, see Figure 12.

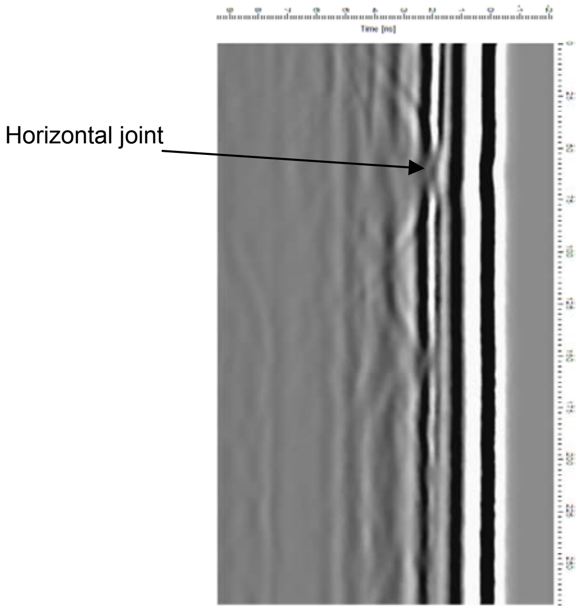
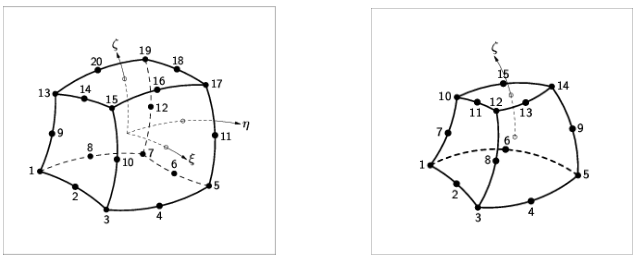


Figure 11. Vertical radar profile

The three dimensional geometrical models of the balcony and the pinnacles were developed in AUTOCAD. Coarse meshes composed of quadrilateral and prismatic shape solid bricks were defined. This procedure was followed so that refinement of the mesh in DIANA could be easier. The final mesh of the structures obtained in DIANA are indicated in Figure 13.



(a) *(b)*

Figure 12. CHX60 finite element – brick, 20 nodes and (b) CTP45 - wedge, 15 nodes

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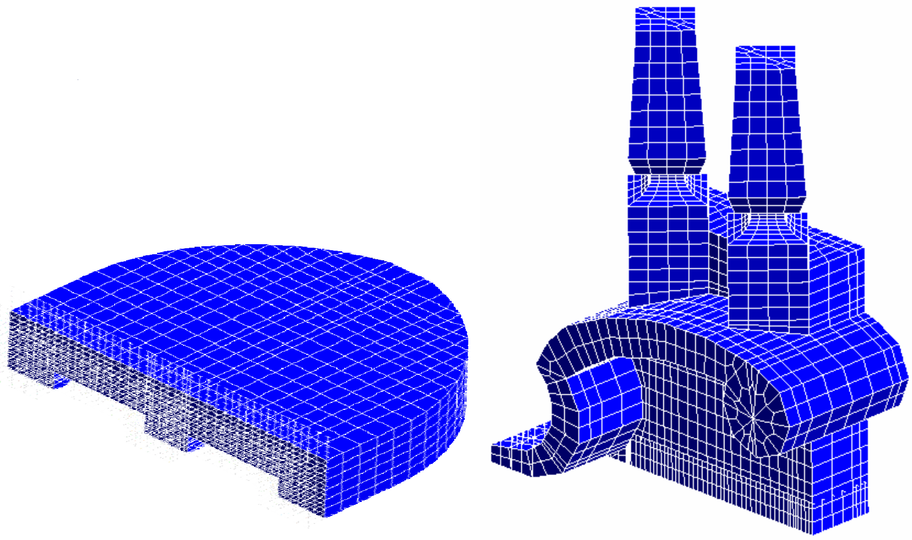

(a) *(b)*

Figure 13. Final mesh and the boundary conditions for (a) the balcony and (b) the pinnacles

The boundary conditions considered for the salient structural elements under analysis aim at simulating the real connection conditions of these elements to the façade. Thus, for the balcony, the constraints were defined in the surface corresponding to the edge connected to the wall. The constraint degrees of freedom were X, Y, Z translations, to describe the actual fixed connection, see Figure 13(a).

For the pinnacles, the constraints were defined at the basis of the column and also of the shell, considering the X, Y, Z translations restrained, see Figure 13 (b).

The material used in the construction of the cathedral and namely of the façade is a two mica medium grained granite. It is believed that it has a high weathering degree, taking into account its yellow colour and its visible superficial detachments.

Based on the simplified petrographical description of the granite used for construction, the mechanical properties adopted were similar to the properties found in Vasconcelos [2]. This author carried out an extended experimental characterization on distinct types of granite with different petrographical characteristics. The mechanical properties considered for weathered two mica medium grained granite are summarized in Table 1.

Table 1: Mechanical properties considered for the granite

f_c (N/mm^2)	E (N/mm^2)	f_t (N/mm^2)	G_f (N/mm)
60.40	15008	1.56	0.234

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The compressive strength, f_c, and Young modulus, E, were obtained from uniaxial compressive tests, whereas, the tensile strength, f_t, and fracture energy, G_f, resulted from direct tensile tests. The stress-displacement diagram exhibiting the tensile behaviour of the granite for different specimens is presented in Figure 14.

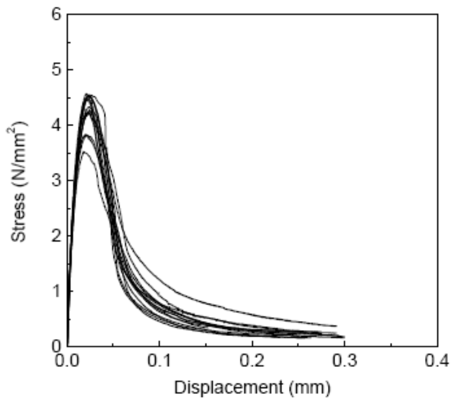

Figure 14. Stress-displacement diagrams of granite obtained in direct tensile tests, Vasconcelos [2]

Both elements, the balcony and the pinnacles, were subjected basically to their own weight.

For the Balcony, the weight was defined as an external load pressure acting on all upper horizontal surface. The loads were calculated by multiplying the volume of the slab and ribs by the dry density of the granite of about 2600 kg/m^3. The weight of the massive balustrade was also applied to the balcony, as element pressure loads in correspondence with the columns.

Due to its complex shape, the own weight of the pinnacles was calculated automatically by the program being given the information about the dry density of granite (2600 kg/m^3) and the gravity acceleration of 9.81 m/s^2.

4.1. –Linear analysis - isotropic material approach

In this section, an overview of the results obtained in the linear elastic calculations is given. This is important so that the initial condition of the structures and the understanding of the stress and strain field distributions and, thus, the detection of the possible critical points where maximum principal stresses occur (feasible cracked zones) can be identified. For the linear analysis only information about Young modulus and Poisson‘s ratio was required.

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The stress distribution on the Y global direction, the maximum principal strains, the minimum principal stresses on deformed mesh and the stress diagram for the linear elastic approach are displayed from Figure 15 to Figure 19.

From the normal stress distribution, it is observed that the maximum values appear at the support's section, where maximum bending moments occur. It is noted that in fact the structure behaves as a cantilever.

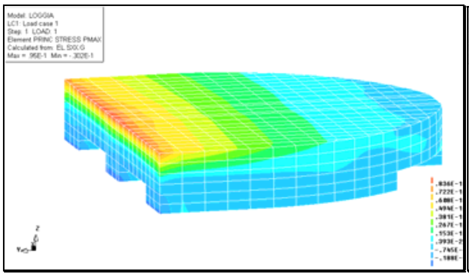

Figure 15. Syy stress distribution

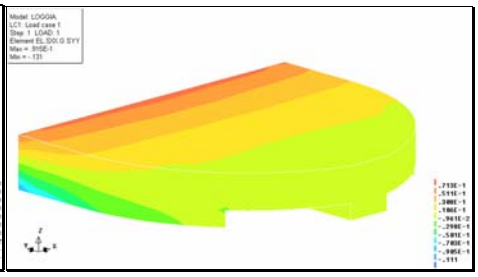

Figure 16. Maximum principal stresses

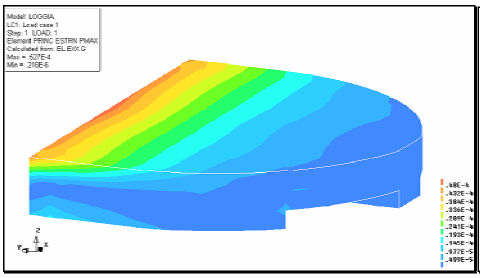

Figure 17. Maximum principal strains

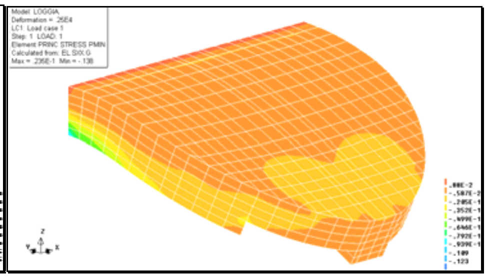

Figure 18. Minimum principal stresses

As expected, the normal stress diagram along a cross section defined at the support is linear, see Figure 19. Tensile stresses occur at the upper part of the section, whereas the maximum normal compressive stresses occur at the lower fibre.

From the distribution of the maximum and minimum principal stresses it can be seen that low values of both tensile and compressive stresses are present, when compared with the mechanical strengths.

Similarly to the Balcony, also for the pinnacles, the linear analysis is performed to emphasize the actual state of the salient elements of the cathedral's façade and also as a preliminary step for running the nonlinear analysis. The results, in terms of displacements, stresses and strains, can be seen from Figure 20 to Figure 24.

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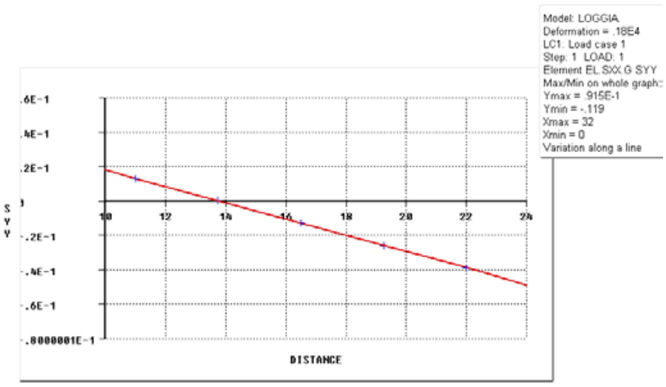

Figure 19. Stress diagram in the critical section

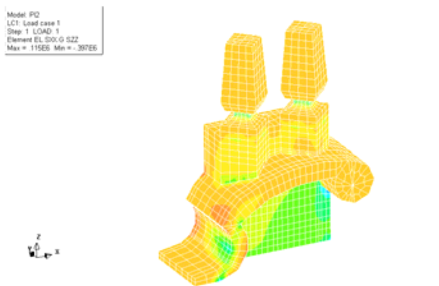

Figure 20. S_{zz} stress distribution

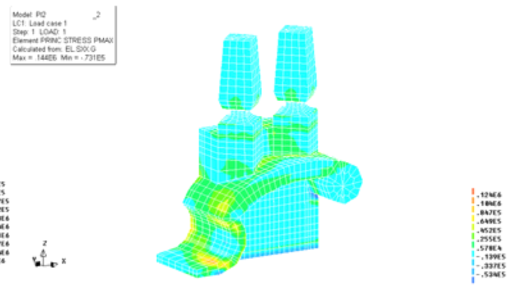

Figure 21. Maximum principal stresses

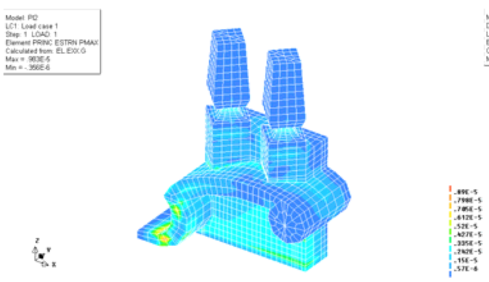

Figure 22. Maximum principal strains

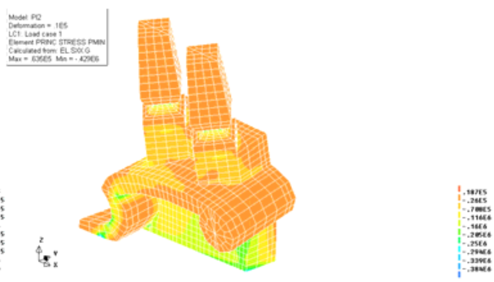

Figure 23. Minimum principal stresses

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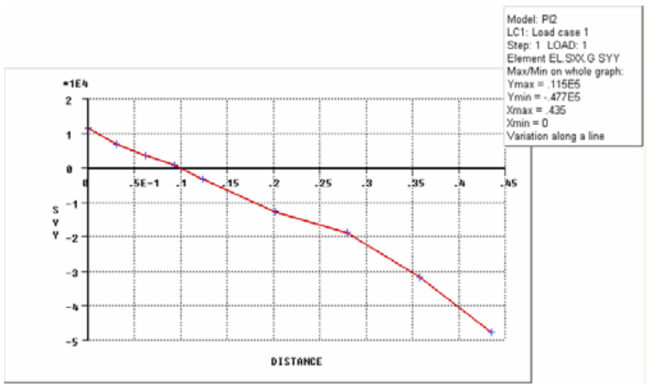
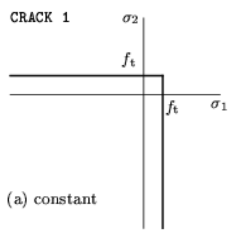


Figure 24. Stress diagram in a critical section

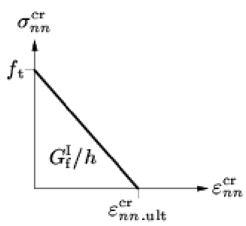
4.2. Non linear analysis

From the results of the linear analysis and based on the mechanical properties of granite, it is possible to observe that material nonlinearity should occur due to tensile cracking. In fact, similarly to concrete, the granite exhibits a much lower tensile strength than the compressive strength.

Therefore, the failure is expected to occur due to tensile stresses that might occur in the upper fibres of the balcony and in the shell belonging to the pinnacles substructure. Thus, for modelling the behaviour of the granite, a smeared cracking model was considered [3]. Cracking is specified as a combination of tension cut-off, tension softening and shear retention, [4]. The fundamental feature of the decomposed crack model is the decomposition of the total strain ε into an elastic strain ε_e and a crack strain ε_{cr}: $\varepsilon = \varepsilon_e + \varepsilon_{cr}$ [5]. Granite is modeled as a combination of constant tension cut-off criterion, linear tension softening and constant shear retention, see Figure 25. Mode I fracture energy is released in the element if the tensile stress is violated and the deformations localize in the element, depending the results on the mesh refinement [5].



Constant tension cut-off



Linear tension softening

Figure 25. Failure criteria granite used for nonlinear analysis of the structures

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Balcony

The relation between the load factor and the maximum vertical displacement obtained at the border of the balcony is shown in Figure 26. The onset of the nonlinear behaviour occurs for a load factor of about 1.8, being the cracking localized in the upper edge of the support's section, see Figure 27 and Figure 28, where the principal stress and strain distributions associated to cracking are displayed. For the tensile strength of granite, the load factor is 4.8. This factor represents the scalar that multiplies the gravity load, indicating the safety level of the structure. Note also that the vertical displacement (2.5mm) is rather reduced. This state corresponds to tensile and compressive maximum stresses of respectively 2.79 and 14.6 MPa. It is important to stress that the compressive stress obtained in the nonlinear analysis is clearly below the compressive strength adopted for the granite, which means that the linear model under compression is valid, see Figure 29. Given the weathered state of the granite, an additional nonlinear analysis was performed considering a tensile strength of granite of about 0.5MPa and a fracture energy of about 0.078N/mm, (corresponding approximately to safety partial coefficient for the material γ_M of 3.0). As expected, the initial behaviour is similar but a considerably lower load factor of 1.8 is obtained, see Figure 26. This means that considering 1/3 of the tensile strength, a load factor 80% higher than the own weight is still achieved. For this situation, the maximum values of the tensile and compressive stress are respectively 0.85MPa and 5.33 MPa.

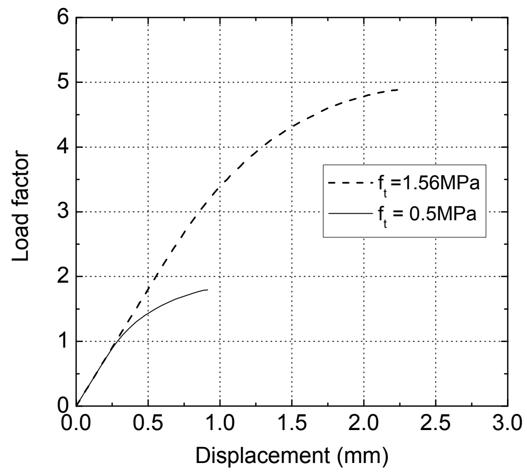

Figure 26. Relation between the load factor and the displacement at the free edge of the balcony

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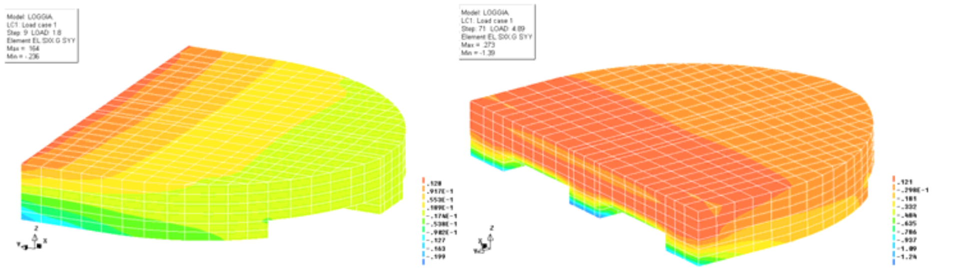

Figure 27. S_{yy} stress distribution for the first crack stage and for the failure stage of the balcony

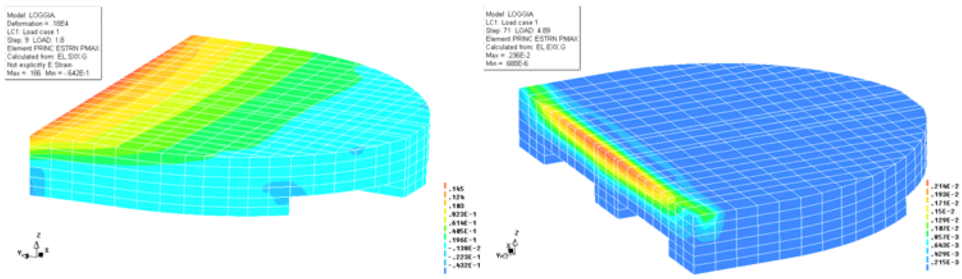

Figure 28. Maximum principal strains for the first crack stage and for the failure stage of the balcony

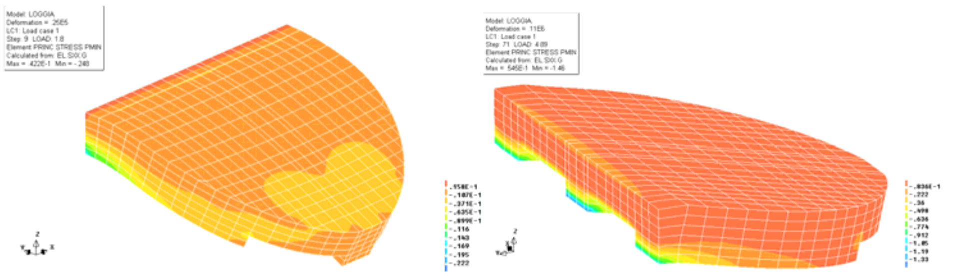

Figure 29. Minimum principal stresses and the deformed shape of the balcony for the first crack stage and for the failure stage

The normal stress distribution at the support's section that crosses the central rib corresponding to the collapse state obtained with f_t=1.56MPa is shown in Figure 30. It is observed that there is possibility of tensile stress transfer in the upper fibre.

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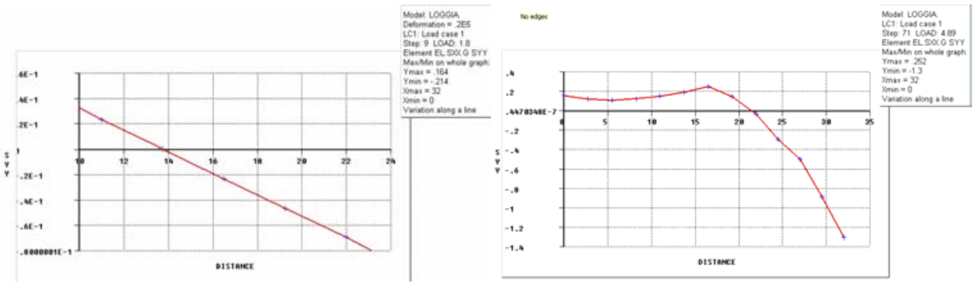
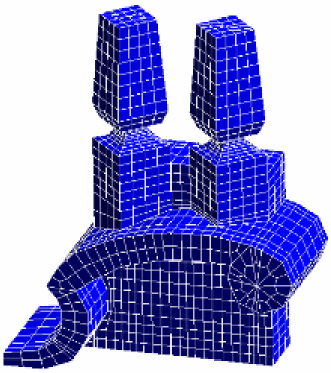


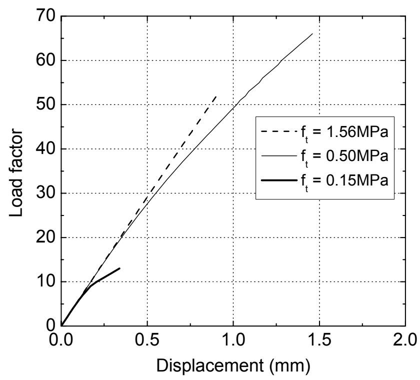
Figure 30: Stress diagram in the critical section for the first crack stage and for the failure stage for the balcony

Pinnacles

The relation obtained between the load factor and the control displacement based on the tensile strength of granite of 1.56MPa is displayed in Figure 31. An additional nonlinear analysis based on a tensile strength of 0.5MPa was also carried for the pinnacles. It can be seen that the response of the pinnacles exhibits reduced nonlinearity when the tensile strength of 1.56MPa is considered, see Figure 31. The high load factor found for both conditions results from the lower level of stresses associated to the own weight.



(a)



(b)

Figure 31. – Results of the nonlinear analysis of the pinnacles; (a) vertical control displacement; (b) relation between the load factor and the displacement control

The nonlinear behaviour of the structure for a tensile strength of 0.5MPa is more evident, which is attributed to some cracking presented by the structure, see Figure 32a. Aiming at evaluating the collapse mode, a nonlinear analysis was carried out taking into account a tensile strength of 0.15MPa. It can be seen that the collapse of the structure occurs in the zone of separation between the support and the

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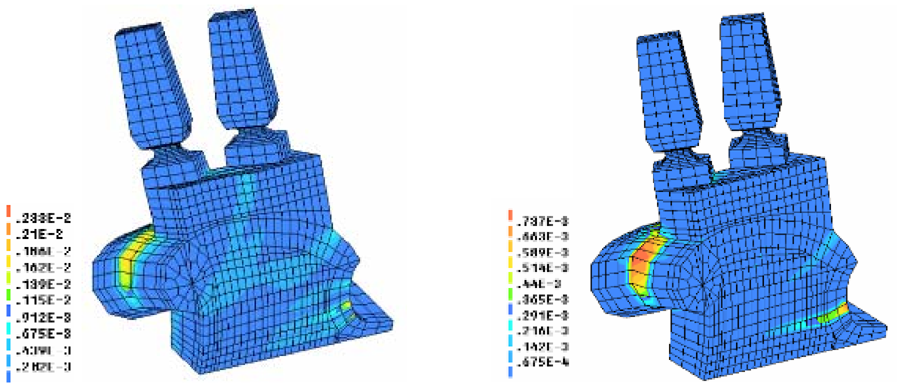
decoration of the pinnacles after the crack opening at the right lower corner of the support, see Figure 32b.



(a) (b)

Figure 32 – Plastic strains for the last load increment: (a) f_t=0.5MPa; (b) f_t=0.15MPa

In the collapse mode the cracking initiates in the right lower corner. As the load factor increases, the crack at the lower corner stabilizes and there is crack propagation between the support decoration of the pinnacles and the lower support. This collapse mechanism can also be observed through the comparison of the force-vertical displacement diagrams from different points located in the crack zones. The displacement of node 12388 exhibits a much more slow increase than the node 6976, see Figure 33.

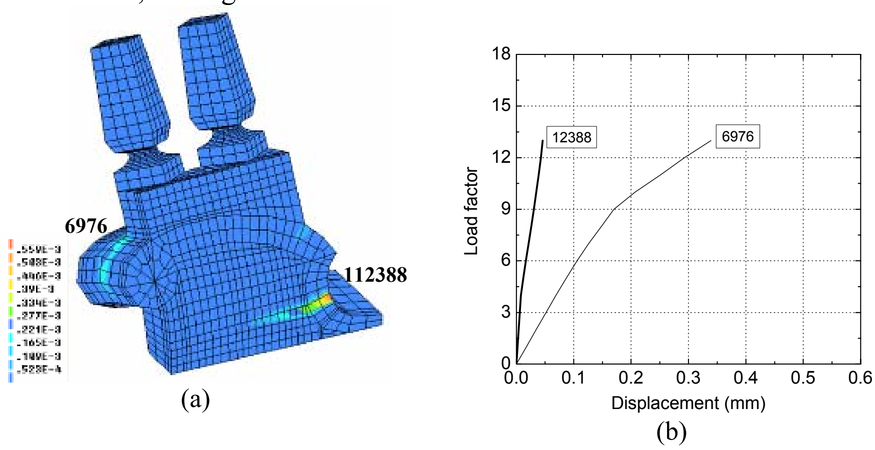


Figure 33 – Analysis of results of pinnacles for a tensile strength of 0.15MPa: (a) plastic strain distribution and control nodes (load factor 10) (b) comparison of force-displacement diagrams for the different cracked zones of the structure

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It is important to stress that even for a much low tensile strength, the load factor is considerably high (higher than 13.0). This allows concluding about the safety of the structure even if the granite presents high levels of deterioration.

5. CONCLUSIONS

The assessment of the stability conditions of the salient elements of the facade of the cathedral of Porto was based in two steps: (a) investigation on the presence of steel connector between stone ashlars and decorative elements and (b) numerical simulation of the balcony and pinnacles.

From the visual inspections it was seen the existence of steel connector between several elements of the façade, having some of them signs of corrosion. From radar inspection, it could be observed that is possible the existence of steel connector into the decorative elements.

The numerical simulation allowed verifying that the maximum tensile and compressive stresses are very low. The load factors resulting from the multiplication of the own weight of the structure are considerably high and in particular for the pinnacles, which guarantees their stability. It should be noticed that the adopted models are homogeneous and the boundary conditions are adequate.

Even so, considering the deep degradation of the granite, it is recommended a surface treatment of the stone, consisting in cleaning the superficial layer and water-proofing the granite surface, so that the polluting factors not to have access to the structure of the stone.

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Performance of the experimental road pavement sectors realized with asphalt mixtures stabilized with various fibers and improved bitumen on National Road NR 17, Vatra Dornei - Suceava

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Summary

This paper presents the evolution in performance of some experimental road pavement sectors realized with asphalt stabilized with various fibers and improved bitumen on National Road, NR17.

A synthesis of the monitoring results, obtained during a period of more than five years is presented here with the appropriate conclusions and recommendations facilitating the implementation of tested technologies in the process of road rehabilitation programs in Romania.

KEYWORDS: road sector, performance, improved bituminous binder, modified bitumen.

1. INTRODUCTION

Since 1995, the National Road Network of Romania became subject to a massive rehabilitation program conducted by the Ministry of Transport using various indigenous and foreign findings.

To make better usage of these resources, the Romanian Center for Road Engineering Studies and Informatics-CESTRIN and the Academia represented by four major Universities involved in civil engineering, road research. & pavement technologies, worked together to implement and apply into practice these new technologies. This study displays the performance data collected on various levels of traffic on different road pavements. The representative samples, based on the improved asphalt mixture, have been tested on the Accelerated Circular Road Track Facility-LIRA, existing at the Technical University "Gh. Asachi" of Iasi, in parallel with the operation of similar experimental sectors realized on the existing road network.

This paper presents a synthesis of the monitoring results obtained during a period of more than five years including the appropriate conclusions and

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recommendations for implementing tested technologies in the frame of ongoing road rehabilitation programs in Romania.

2. STUDIES

2.1. Description of the experiments

Fibers and additives for the increase of binder bonding on the aggregates surface have been used for preparing wearing courses, to increase the resistance of pavement in sever climatic conditions (very cold winters with many frost cycles and very hot summers) specific to the Romanian road network.

Asphalt mixtures, stabilized with cellulose fibers and having a higher percentage of aggregates (up to a 75%) have been found to assure an adequate stability of pavement over the summer and improved resistance during the cold winter.

The National Company for Public Roads & Motorways-CNADNR and CESTRIN planed and applied this experiment during the month of august 1999, on National Road NR17, on five experimental sectors. The performance of these experimental road sectors was monitored during a specific time frame, in real traffic and climatic conditions, as mentioned in Table 1[1].

Table 1. Temperature variability on the experimental road sectors on NR17

Year	T_{max}				T_{min}	
	Recorded	Calculated			Calculated	
		$T_{a(max)}$	$T_{s(max)}$	$T_{20(max)}$	$T_{a(min)}$	$T_{s(min)}$
Summer 2000	30.5	28.4	49.8	47.3	-	-
Winter 1999-2000	-	-	-	-	-25.2	-19.9

The temperatures mentioned in Table 1 have the following significance:
- $T_{S(max)}$ is the maximum surface temperature,
- $T_{a(max)}$ is the maximum air temperature,
- $T_{20(max)}$ is the maximum earth temperature at 20cm deep,
- $T_{S(min)}$ is the minimum surface temperature,
- $T_{a(min)}$ is the minimum air temperature.

The following experimental sectors and technologies have been periodically monitored in order to assess their performance at various stages of their life.
- The experimental sector No.1, **km131+800-131+900**: wearing course constructed with Asphalt Concrete BA16 and realized with modify bitumen with the reactive polymer-ELVALOY AM [2].

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- The experimental sector No.2, **km131+900-132+100**: wearing course constructed with Asphalt Concrete BA16 realized with modify bitumen with the reactive polymer ELVALOY AM and stabilized with cellulose fibers type TEHNOCEL 1004 [2].
- The experimental sector No.3, **km132+100-132+300**: wearing course constructed with Asphalt Concrete BA16 stabilized with synthetic fibers type PNA [2].
- The experimental sector No.4, **km132+300-132+400**: wearing course constructed with Asphalt Concrete BA16 stabilized with cellulose fibers type TEHNOCEL 1004 (left lane) and with cellulose fibers type TOPCEL (right lane) [2].
- The experimental sector No.5, **km 132+400-132+700**: wearing course constructed with Asphalt Concrete BA16 and bitumen, treated with additive INTERLANE IN 1400 [2].

Table 2– The composition of the various asphalt mixes used in this experiment [2]

Components	**Sector 1**	**Sector 2**	**Sector 3**	**Sector 4 right**	**Sector 4 left**	**Sector 5**
Chippings size 8-16 mm	39.20	39.10	27.90	39.10	39.10	39.20
Chippings size 3-8 mm	26.10	26.00	27.00	26.00	26.00	26.10
Fine crushed sand size 0-3 mm	18.70	18.60	28.80	18.60	18.60	18.70
Lime filler	9.30	9.30	9.00	9.30	9.30	9.30
Bitumen D80/120	6.60	6.60	7.00	6.70	6.70	6.65
Reactive polymer - Elvaloy AM	0.10	0.10	-	-	-	-
Additive – Interlene IN 400	-	-	-	-	-	0.05
Cellulose fibers type Topcel	-	-	-	0.30	-	-
Synthetic fiber type PNA	-	-	0.30	-	-	-
Cellulose fibers type Tehnocel 1004	-	0.30	-	-	0.30	-
Total (%)	**100**	**100**	**100**	**100**	**100**	**100**

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2.2 Results of the analysis of technical conditions on the experimental road sectors, immediately after construction [3]

- **The first** experimental road sector realized with polymer ELVALOY AM presents an open texture with porous areas located in the middle of road sector and between the wheels tracks on both lanes. The general aspect of the surface concords with the results of the laboratory investigations made on specimens taken from the pavement. The surface does not present major distresses.
- **The second sector** realized with polymer ELVALY AM and TEHNOCEL 1004 fibers presents a similar surface as the first one, only that the porous areas are the largest and irregular in form. These areas are alternating left to right on lengths of about 20m. Laboratory tests have shown that there are many differences in the content of fine aggregate between the specimens. In some porous areas loss of aggregates have been observed.
- **The third sector** realized with PNA fibers presents a surface of about 25sqm with uncoated aggregates alternating left to right. There are also differences between lanes: the one on the right has a smooth surface where the left one has a rough aspect. Furthermore, this sector presents isolated separations of aggregate without major degrades. Right in the middle of the sector on a length of about 25m there are cracks transmitted from the longitudinal joint of existing concrete pavement.
- **The fourth sector** realized with TEHNOCEL 1004 and TOPCEL fibers presents a compact surface. Roughness distresses alternates on both lanes from 50 to 50 m.
- **The fifth sector** realized with additive INTERLENE IN 400 presents a dark asphalt surface, and the aspect of the sector is adequate.

2.3 Results of the technical conditions of the experimental road sectors five years after construction [4]

According to this evaluation made on November 2004, the technical conditions of road sectors have been found as follows:

- **Sector 1**: The evolution of initial porous surfaces was not significant. Some fretting have appeared especially on the area between wheel tracks.
- **Sector 2**: The principal characteristic of this sector was its porous structure with raveling over most of the surface. The main distresses observed on this sector are: submitted, inadequate repairs after the extraction of cores, transversal cracks found frequently, most of them starting from the middle of the road and some longitudinal cracks especially on the area between wheel tracks.
- **Sector 3** - On this sector the surface has pores and cracks on the area between wheel's traces especially on the right lane. The main distresses observed are:

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transversal cracks on both lanes but more on the right side, lane inadequate repairs after the cores extraction on both lanes and some longitudinal cracks.

- **Sector 4** – On this sector the following distresses have been noticed: ineffective repairs after the cores extraction on both strips and some transversal cracks starting from axle with a reduced severity rate.
- **Sector 5** - The sector exhibits a similar surface as the previous sector except some transversal and longitudinal cracks.

2.4. Results of the last examination made by our team in December 2004

Our findings are confirming the previous results recorded by our colleagues of Technical University of Cluj-Napoca [4], as follows from:

- In accordance with Fig.1, Fig.2, Fig.3 from below, the surface on the first sector stabilized with ELVALOY AM polymer is darker and presents less distress in comparison with the second sector realized with ELVALOY AM polymer and TEHNOCEL 1004 fibers. The most frequently degraded areas have transversal cracks where the repairs were made after the cores extraction and longitudinal cracks at the joint between the lanes. Considering last year as an example we can see that the evolution of the cracks transversal and longitudinal shows an increased number, length, and severity rate.

Experimental Sector 1

Figure 1. Raveling between wheels tracks

Experimental Sector 1

Figure 2. Longitudinal cracks & raveling

- The advanced rate of degradation on the second sector realized with ELVALOY AM polymer and TEHNOCEL 1004 fibers is due to the moisture that infiltrates through cracks into the road structure. The longitudinal fissures observed between the lanes on the previous sector continue on this too. The severity of cracks between the lanes on some area has been repaired with asphalt mix. The number and severity of the

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previous distresses have significantly increased as shown in Fig 4, Fig.5, Fig.6 and Fig.7 from below.

Experimental Sector 1

Figure 3. Transversal cracks

Experimental Sector 2

Figure 4. Porous surface

Experimental Sector 2

Figure 5. Raveling

Experimental Sector 2

Experimental Sector 2

Figure 6. Longitudinal cracks

Figure 7. Transversal cracks

- In accordance with Fig.8, Fig.9, sector number three presents a surface with a homogenous aspect and a reduced number of distresses. The severity of cracks, on longitudinal and transversal areas during the same period, has significantly increased. The longitudinal cracks are observed more often between wheel tracks. The longitudinal cracks of the joint between the lanes increased significantly. The fretting is also enlarged and there are areas where these distresses were evaluated to small pot-holes. Some repairs with asphalt mix are also observed on small surfaces. The longitudinal cracks from axle continue on the whole sector's length.

Experimental Sector 3

Figure 8. Transversal cracks

Experimental Sector 3

Figure 9.Longitudinal cracks

- The fourth sector presents a surface with an aspect more homogenous than the previous ones (see Fig.10 and Fig.11). Exception makes the middle of the areas between wheels tracks which are more porous having frettings. In addition, this sector appears to have an increased length and number of cracks with a reduced severity rate.

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Experimental Sector 4 | Experimental Sector 4

Figure 10. Raveling between wheels traces | Figure 11. Sector with good performance

- The fifth sector presents a similar surface with the previous sector except some transversal and longitudinal cracks, as shown in Fig.12 and Fig.13 from below:

Experimental Sector 4

Figure 12. .Sector with good performance

Experimental Sector 4

Figure 13. Longitudinal cracks

We may conclude that taking into consideration real traffic and climatic conditions of the NR17 the investigated experimental sectors exhibit a satisfactory behavior at this stage. It is essential to monitor and to inspect more frequently the performances of these sectors and to perform the necessary repair and maintenance works.

It is envisaged that the monitoring of performance of these sectors to bee continued for at least another five years.

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