Reliability and Performance in Bridge & Transportation Infrastructure Engineering

C. Ionescu, F. Paulet-Crainiceanu, R. Andrei

editors

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Bridges and Transportation Infrastructure Engineering, a highly dynamic research field

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Summary

Bridges and Transportation Infrastructure Engineering has become a very dynamic research field because of its social and economic impact and also because of last years of tremendous developments in other domains that helps evolution of human activity as electronics, computer science, composite materials and so on.

This paper tries underlining the new trends and developments that are revealed through the publications in [1]. Interesting methodologies and techniques are presented and analyzed.

This paper hopes to bring to the reader a global image of some matters at the core of researches in Bridges and Transportation Infrastructure Engineering and, therefore, to lead engineers, students, Ph.D. students, researchers, practitioners and professors to directions that are of interest for civil and academic society.

Keywords: bridges, transportation infrastructure, international symposium, dynamic research

1. INTRODUCTION

In Bridges Engineering as in Transportation Infrastructure Engineering fast changes are underway because the human society needs more and betters means of transportation. Researches done in other fields of activity, as Computer Science, Electronics, Automation, new materials etc., are strongly influencing the domains under study in this paper.

Bringing together the ideas extracted from [1], this paper proposes an overview on what are the current problems that stand in front of researchers in Bridges and Transportation Infrastructure. Very interesting achievements are presented, to the aim of giving to civil and academic society a look on major concerns that absorb the efforts of researchers and practitioners.

The works in [1] are analyzed one by one, emphasizing the accomplishments and the novelties. Conclusions are withdrawn, hoping to help the reader in selecting the parts more important from his or her point of interest.

2. TRENDS AND ACCOMPLISHMENTS IN BRIDGES AND IN TRANSPORTATION INFRASTRUCTURE ENGINEERING

In paper [2], Bota is dealing with the problem of damaged concrete bridges in Timis County, Romania. For the case of a damaged concrete bridge, two possibilities are deeply analyzed: rehabilitation of the existing bridge or construction of a new bridge. The papers is showing that the rehabilitation of the old bridge would be very difficult and at a high cost and, therefore, construction a new bridge is proposed and a new structural system is presented.

The same author, in [3], is describing a part of the large amount of work underway in the city of Timisoara, Romania, regarding the rehabilitation of transportation infrastructure in that city. The rehabilitation was needed because of traffic intensification and weather condition during many years. Four bridges rehabilitation is shown. A special attention is given to the historic Dacilor and Mihai Viteazu Bridges. Also, the choice of a method for rehabilitation of tramway on bridges is studied and adopted.

The influence of railway dislevelments on dynamics of bridges is the main topic of the paper [4]. The authors, Bucur, C. and Bucur, V.M. study six types of bridges. Parametric studies are aiming to obtain the dynamic magnification factors as functions of the amplitude and length of dislevelments. Finite element models for structure and vehicle are shown along with the hypotheses for calculations. The work methodology is explained. For the studied cases results are presented and commented. Favorable and most unfavorable cases are underlined.

The reference [5] is presenting the case of a rehabilitation work on a bridge. The author, Proca, G.E. is treating the problem of the environment protection quality management during the bridge repairing. The paper presents the geographical condition for the bridge location. Long with the construction work, environmental protection work is explained. Pollution sources for the air, soil and water are identified and quality control is performed. The importance of the environment protection work together with the technical is highlighted.

In [6], Hildebrand and Nunn present the European cooperation in the field of accelerated load testing (ALT) for analyzing pavement behavior. The aim is to harmonize the European countries' ALT effort under the European Comission's COST (COoperation in the field of Scientific and Technical research) initiative. After presenting the ALT and COST action 347, referring to pavement design, research and maintenance, the scientific program and an inventory of European

ALT facilities is performed. Previous and current ALT research work is analysed. The common code of good practice for application of ALT is shown. The future use of ALT, dissemination and international cooperation in the field of ALT under COST are discussed. Finally, accomplishments of COST 347 are revealed. In Appendix, tables and figures illustrate and support the text.

Stryk, Pospisil, Korenska and Pazder, [7], are dealing with the modal analysis and acoustic emission methods for determination of prognosis regarding the future behavior of bridges and also determination of residual life for those structures. The authors are proposing a method for finding the characteristic frequencies that are generated by the corroded reinforcement of concrete bridges, also called “unhealthy sound”, from the Acoustic Emission (AE) recorded on bridges. The state of the art of this domain of research is performed. The monitoring equipment (data acquisition system with four channels, computer, software, sampling card, preamplifiers, amplifiers and piesoelectrical sensors,) is presented. For the propose of mastering the method, laboratory work was also performed. In-situ work was also performed on the superstructure of a reinforced concrete bridge under reconstruction. The proposed method is shown effective and useful for supplementing the other diagnosis methods.

In his paper, [8], Pospisil is presenting the facilities and the technology researches for Transportation Infrastructure domain underway in the Transport Research Center (CDV) in Czech Republic. Firstly, the Geotechnical Laboratory Testing Field facility is shown. Details and testing possibilities (plate test, dynamic load test, penetration test etc) of using this facility are revealed. Results for deformation modulus for various soils type are presented. The use of geosynthetics for incresingthe bearing capacity is also studied and analyzed. A second part of the paper is dedicated to self-compacting concrete studies and the third part is related to non-destructive tests on bridge structures. Conclusions are underlining the tremendous research activity at CDV.

Based on Prompt Neutron Gamma Activation Analysis (PGNAA), Peticila, Tripadus and Craciun, [9], setup and use a methodology for determination of the carbon over hydrogen ratio in bitumen. The spectra are obtained through the records of the output from bitumen samples penetrated by neutrons from a Am-Be source. Mathematical processing of spectra is showing very good results in determination of carbon/hydrogen ratio. After that the ration is useful in obtaining other bitumen characteristics, as colloidal index and, therefore, judgment on the quality and properties of the bitumen could be issued.

In [10], Proca, M. and Proca, G.M. are considering some aspects of monitoring the behavior of bridges through the use of surveying. The study is occasioned by the rehabilitation works underway for a bridge over Siret River. The rehabilitation work is described thoroughly. Using precise surveying the behavior of the bridge and the rehabilitation work are evaluated. In conclusions, the necessity and efficiency of periodically checks on behavior of bridges through topographic measurements are underlined.

Gavris is proposing a numerical method for obtaining the influence surfaces for the bridge decks [11]. The method is based on Finite Strip Method, a method based on variational principles and that insures a faster computation time and a good accuracy. The theoretical background is shown. A computer program was made and a numerical example is presented, in order to evaluate the efficiency of the method. Comparisons with other methods of computation reveal very good results of the proposed procedure.

The paper [12] is dealing with the general problem of research and technology for highways that have large impact on the social and economics life of Romania. The author, Andrei, is presenting a short history and tradition of Romanian roads and bridges. Then he is detailing the elements of Romanian public road network from which one third is classified as European roads. In this context, the Strategic Highway Research Program (SHRP), Romanian Long Term Pavement Performance Program (RO-LTPP), SHRP/Superpave Laboratory etc. are considered milestones of the dynamics of the field. Then, more details and analysis are focused toward the Romanian participation to SHRP/Superpave Laboratory in Bucharest. The development of a climatic database for Romanian transportation system is shown. Aspects of researches done at facilities in Technical University of Iasi under RO-LTTP are revealed. Creation of a virtual South-East European Forum for Superior Pavements is presented along the goals of this forum.

Andrei is presenting a methodology for assessment of road pavement condition [13]. The main tools for this methodology are the concepts of chance, change and entropy. These tools are expresses in terms of probabilities, dynamics and system's entropy. After a theoretical background presentation, the principles of modeling natural systems applied to road pavements are revealed. Then, each of the three concepts is analyzed. Conclusions and recommendations for further studies are also underlined.

A problem concerning the plane plates' deflection coefficient is analyzed by Jantea and Varlam in [14]. The paper studies the case of plates acted by forces in their median plane. Firstly a theoretical energy based approach for determining the critical load and using series decomposition is proposed. For a particular action case and different ratios of the plate dimensions, numerical results are computed. A comparison with values for the deflection coefficient given by a Romanian code is performed. Significant differences are found for larger ratios between the length and height of plates. However, the differences found in the code is assuming a more cautious path (i.e. the critical load is smaller than the theoretical one).

In [15], Blagoi and Vlad are dealing with the design of culverts. A fast and economical method for design of culverts is proposed. In introduction the general conditions imposed for culverts are revealed. Then a classification of culverts is performed. Based on the concept of reliable hydraulic design, the mathematical procedure of semi-forced rectangular culverts design is presented. A computer program is conceived and numerical results are shown. Nomograms useful for fast and accurate design are issued.

Zarojanu and Boboc, [16], are proposing a probabilistic approach for evaluating and forecasting the technical condition of flexible-semi-rigid road pavements. Firstly the indexes of pavement layer distress condition are presented. The distress severity degrees and frequency of occurrence are shown. Then the way to calculate the evaluation indexes is presented along with the method for characterizing the pavement condition. A case study is showing practical data of two national road sectors and the statistical results obtained from the data. The typical statistical distributions revealed are used at the base for programming and prioritization of the maintenance work.

A study on the amplification of the active pressure on retaining walls due to seismic action is presented by Moga in [17]. The paper shows the elements and parameters that interact during a seismic event in relation to retaining walls. Theoretical background for calculation of the seismic pressure is revealed and commented. Intensive parametric analyses are conducted. Depending on the seismic coefficient corresponding to location of retaining wall and depending on geometrical parameters of the wall and soil characteristics the results are showing significant amplifications of seismic pressure compared to static case (close to two times in some cases).

3. CONCLUSIONS

In the field of Bridge and Infrastructure Engineering, there is a tremendous and diverse activity underway. This paper analyzed the technical and scientific work done in this domain of constructions as it is revealed by the papers in [1].

The described analysis is related to almost all directions as: bridges design and rehabilitation, bridge monitoring, railways transportation, pavement monitoring and rehabilitation, roads construction and protection, environment protection during transportation infrastructure works and maintenance, European cooperation, etc.

As a consequence, it is important to continue researches and meetings of people involved in Bridge and Transportation Infrastructure for exchanges of knowledge and concerns and for assuring a robust future for this important domain of the human society.

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A new structure with embedded girders

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Abstract

The structure, made in the first decades of the last century, is set in the Timis county, near Timisoara. The degradations and low concrete classes, out of which the elements of the superstructure and substructure were realized, were determined by expertise the structure. It should be considered that the degradation process of the structure has evoluted in a time span of 9 years passed after the moment the expertise was done. We have considered two alternatives for the old structure: either rehabilitation and consolidation of the structure and substructure, or the realization of a new viaduct of discharge, placed in the same emplacement as the existing structure or in a new one.

1. GENERAL ASPECTS

In the world economical context and also in the national one, when the mobility of the individual is being emphasized and when the assurance that he is provided with all the facilities of the 3rd millennium is very important, realizing terrestrial communication ways in order to offer the users the possibility of fast, safe and comfortable traveling, is absolutely justified.

In order to do this, the passing structures like the footbridges, bridges or overpasses have to fit in the spirit of the world conception in this field. This means that a merge of robustness and durability with aesthetics and functionality is imperative.

A crossing structure well designed and built/rehabilitated under technical accuracy will provide during exploitation, safety and low costs for the users on one hand, respectively for the administrator on the other hand.

2. ACTUAL SITUATION

The structure, made in the first decades of the last century, is set in the Timis county, on DJ 592 at km 14+011, near Timisoara and assures the continuity of the county road over the right major bed of the Timis river, between the localities Albina and Cheveresul Mare, playing the role of a viaduct for discharging high floods.

The statically system of the viaduct is a frame with four spans as follows -16,50 m + 2 x 2,50 m + 16,50 m (Fig.1). The cross section consists of four reinforced concrete beams (C6/7,5) having a width of 30 cm (Fig.1). In the knots the girders have straight haunches. The height of the girder varies from 1,38 m in the field up to 1,98 m at the piles, respectively up to 1,83 m at abutments. The cross rigidity is assured by means of cross beams on the abutments and with reinforced concrete ribs in the field (Fig.2). The abutments and the piers are made of concrete belonging to the class C 2,8/3,5, indirectly founded on 15 piles of reinforced concrete having a square section of 30 x 30 cm (Fig.3).

Fig.1 Front view

Fig.2 Intrados view

Fig.3 Pier founded on piles

The concrete classes, out of which the elements of the superstructure and substructure, mentioned earlier, are realized, were determined by expertise the structure. It should be considered that the degradation process of the structure has evoluted in a time span of 9 years, passed after the moment the expertise was done

(Fig.4). At this moment the gauge of 7,40 m includes a runway of 6,00 m and two footways of each 0,70 m (Fig.5).

Fig.4 View at intrados

Fig. 5 View towards Albina locality

When we studied this case we have considered two alternatives:

A. Rehabilitation and consolidation of the structure and substructure, the correction of the major bed of the Timis River in the section of the bridge, the elimination of the scouring and also of the causes which led to them.

B. The realization of a new viaduct of discharge, dimensioned from hydraulic point of view, placed:

a. in the same emplacement as the existing structure

b. in a new emplacement, on the discharge direction of the water at very high levels.

1. Alternative A. – Rehabilitation and consolidation of the structure of the existent viaduct

Rehabilitation and consolidation of the structure impose a higher level of loading and the enlargement of the gauge. For the fulfilling of these requests the following works have to be done:

- creating a traffic alternative due to the fact that the needed works can not be done under traffic;
- the consolidation of the foundations, because the bearing capacity of one pile represents only 50% of the necessary - involves the following works:
  - removal of the rock dyke from the piers and the realization of a technological island;
  - execution and launching of a reinforced concrete caisson through manual digging in extreme narrow places;
  - the launching level of the caisson is limited by the stability of the piles and of that of the hole structure, considering the high degradation level of the concrete from the piles and that of their reinforcement;
  - in order to obtain an efficient solution the bottom area of the piles should be injected;
  - the realization of the caisson leads to a significant diminution of the flow section and amplification of the scouring phenomenon, which is very pronounced already (aprox. 3 m).
- the consolidation of the elevation of the substructure, as the concrete quality is absolutely improper C2,8/3,5 in comparison to the minimum that is necessary C16/20 – requests the following works:
  - consolidation and matching of the elevation sections with the new loadings through:
    - coating with reinforced concrete;
    - or carbon fiber (unpermitted solution due to the low class of the concrete);
  - expansion of the support area in order to take-up the loading from the hardened superstructure with a enlarged gauge, by executing an additional girder on the top of the pier, which should work together with the existent structure of the pier through some bars of high resistance or prestressed reinforcement;
  - or entire replacement of the elevation which involves its separation from the superstructure, the provisional support of the superstructure with an adequate scaffolding and the renewing of the boundings superstructure – substructure;

- the consolidation of the superstructure, as the bearing capacity is unsatisfactory, the concrete quality is absolutely inappropriate C6/7,5 in comparison to the requested minimum C16/20 and the gauge is insufficient – involves the following works:
    - realizing an embedment structure with additional beams that will rest on the rehabilitated substructures, so that the gauge requested by the normatives in effect can be reached;
    - consolidation of the existing beams through:
        - coating with reinforced concrete;
        - or carbon fiber (unpermitted solution due to the low class of the concrete);
    - by consolidating the structure, the empty weight is growing significant, phenomenon which leads to general negative effects on the behavior of the substructure;
    - due to serious degradation of the reinforcement and the concrete (corrosion and carbonation), the elements belonging to the superstructure cannot be taken into account when estimating the bearing capacity.

The consolidation solution involves a negative change of the architectonical aspect of the bridge and implicitly has as consequence the loss of the identity of the existing bridge.

2. Alternative B. – Building of new discharge viaduct

This alternative supposes the realizing of a new structure in another location, the execution of a new traffic alternative, demolition of the old bridge, rearrangement of the earthwork in the area of the demolished bridge and the reestablishing of the riverbed. The location was chosen based on photographic recordings of the water flow in the major bed at high floods in correlation with the configuration of the land, showed using topographical measurements. In this situation the direction of the discharge of the waters will be perpendicular on the bridge axis, in contradistinction to the actual situation, reducing that way the level of the local scouring at the substructure elements.

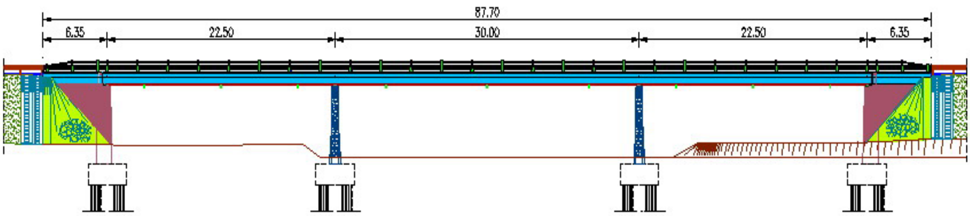

Fig.6 Front view

Considering the amplitude, complexity and the duration of the works development for the designed structure, the traffic continuity will be assured on an asphalted traffic alternative realized upstream. This provisional alternative has one traffic lane and the traffic is being directive by traffic lights.

The static schema of the designed bridge is a continuous concrete slab with three spans: 22,5 m + 30,00 m + 22,5 m (Fig.6).

In order not to modify the discharge conditions of the h high floods, when designing the new structure, the level of the intrados of the old viaduct was maintained and an enlarged outlet was realized by reducing the substructure number and extending of the total length to 88 m.

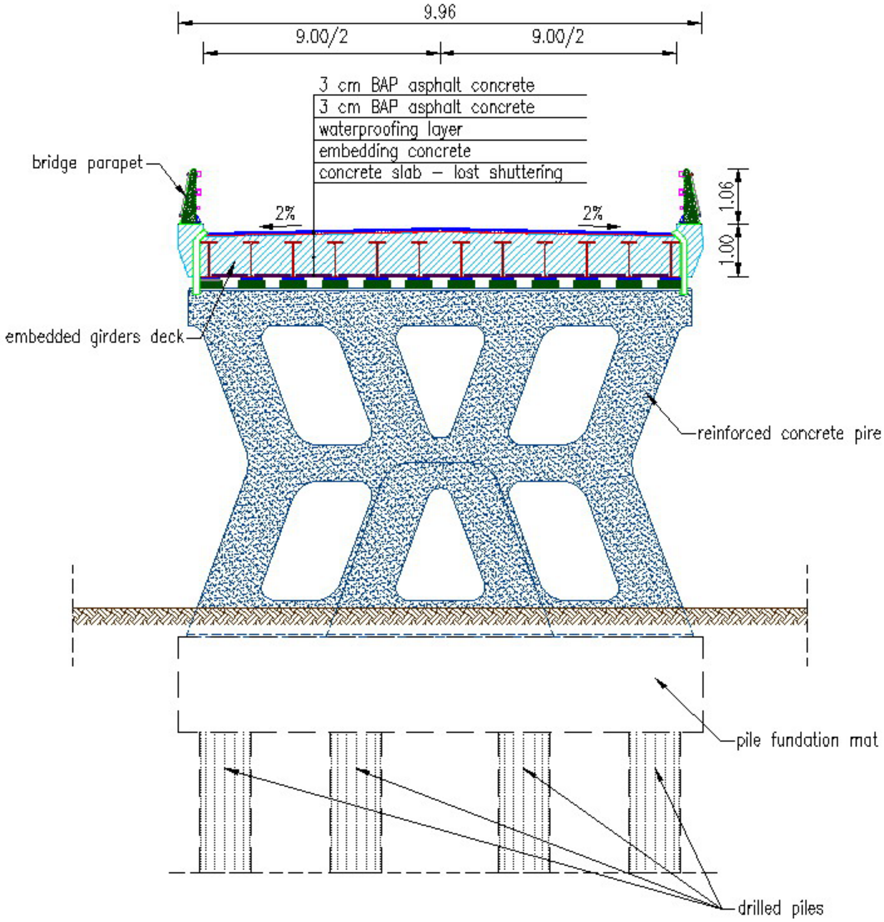

Fig.7 Cross section. 1st alternative

For the continuous superstructure, the solution with 12 HE beams is being suggested. The embedded beams are 75,40 m long. For the sustaining of the embedding concrete, when casting a lost shuttering of precasted slab will be used (Fig.7, Fig.8).

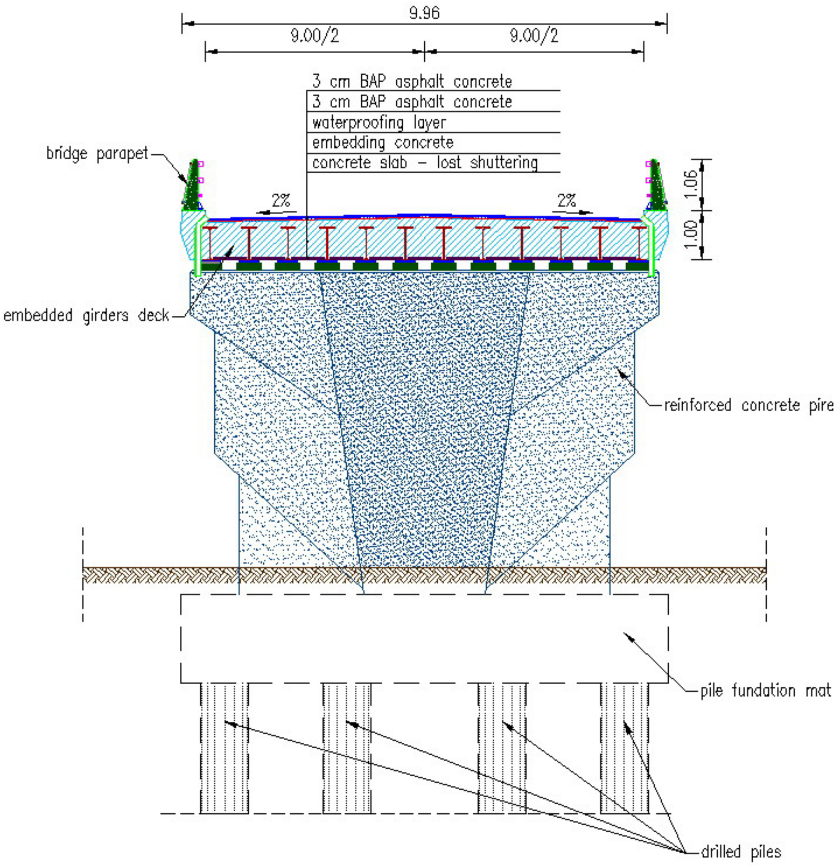

Fig.8 Cross section. 2nd alternative

The total width of the superstructure is 9,96 m realizing a gauge of 9,00 m, specific for the bridges placed outside of localities (without footways) (Fig.7, Fig.8).

The runway has longitudinal reradiating directional pavement marking and luminescent buttons having a free distance of 2,00 m.

The roadside obstructions on the bridge, protected against corrosion by zinc coating, is conceived in a metallic solution, provided with structural elements in order to offer safety for the traffic and pedestrians.

The discharge of the meteoric waters from the bridge is achieved through gullies, respectively by means of side ditches at the extremities of the viaduct.

The bearings are made of reinforced neoprene and are put under each metallic beam.

The substructure consists in two abutments and two piers.

The elevation of the abutments is going to be made of plain concrete in classical solution, having reinforcement only at the joint elevation-foundation.

The two piers have the elevation made of reinforced concrete giving the opportunity for significant slenderness. From architectural point of view, we suggest two alternatives for realizing of the elevation of the piers.

Due to hard foundation conditions, the option was for indirect foundations on piles with a large diameter (columns).

Because the statically system is being chosen as a continuous structure with three spans, the expansion joints are used only at the extremities of the structure (at the abutments).

Considering the high difficulty level concerning the realization of the consolidation of the existing structure, regarding the technical and technological solutions possible to be put through, as well as the controvertible efficiency of these solutions dealing with a very high degradated existent concrete having also a low quality, it is considered that the entire replacement of the old structure is necessary.

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Rehabilitation of tramways and bridges in Timisoara

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Abstract

The paper presents some results of a small part of a large rehabilitation programm applied both to tramways and to carriageways and bridges too. Typically for Timisoara town is the oldness of the crossing structures: 80 years old. The four bridges submitted to this process of rehabilitation are different regarding the statically structures and cross section too. It was necessary to redesign the gauge on the bridges and the oldest one needed consolidation. For the first time in Romania, for the fastening of the rail on the bridge, the system ICOSIT from Sika was adopted.

1. GENERAL ASPECTS

The Timisoara city has the benefit of important external financing for the modernizing of its tramways. At the same time with these projects, the city hall of Timisoara is financing the rehabilitation of the streets affected by the works for the modernizing of the tramway and implicit that of the bridges, which assure the connection between these streets over the Bega channel.

Regarding this, we had the privilege to participate at the elaboration of the technical solutions for the redevelopment of the streets, as well in their geometry as also as road structure, of the bridges and of the underground network affected by the works for the rebuilding of the tramway platform. This meant the replacement and supplementation, where needed, with new capacities of the underground network (electrical, water-channel, natural gases, telecommunication, district heating).

In the actual state the streets, having 2 or 4 traffic lanes, have the carriageway made out of concrete asphalt (Revolutiei 1989 Av., Dr. I. Nemoianu str., A Saguna str., Calea Dorobantilor etc) or granite block pavement (Mihalache str., partially Dacilor str. etc).

The carriageways were designed in correlation with the specific environment and the road structure was adapted to the existing traffic and also to that in perspective.

The use of the granite block pavement on some streets is part of the architectonical concept regarding the arrangement of the Traian Square and the areas next to it.

2. THE REHABILITATION OF THE BRIDGES

The rehabilitated lines of the tramway are passing the Bega river in 4 sections, namely: Mihai Viteazu Bridge between Calea Dorobantilor and A. Saguna street, Dacilor Bridge connecting the streets Kogalniceanu and Dacilor, Decebal Bridge between Revolutiei 1989 Boulevard and 3 August 1919 street, respectively Traian Bridge on the 16 Decembrie 1989 Boulevard.

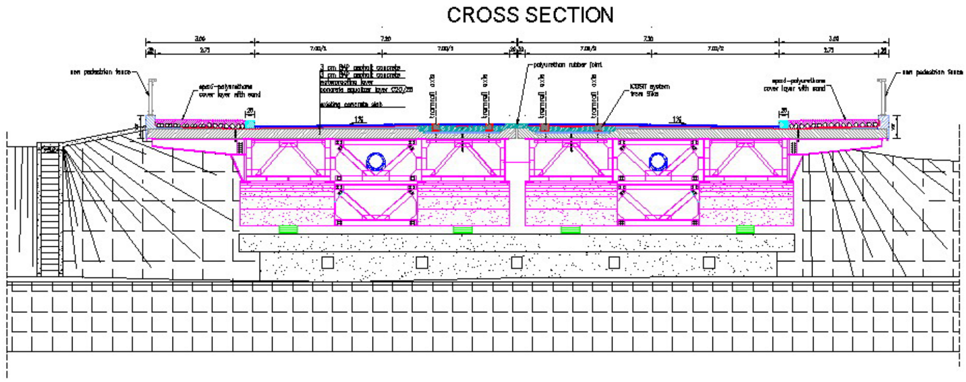

Fig.1 - Mihai Viteazu Bridge

Under the influence of the traffic, which is extremely intense and heterogeneous (car and tram), due to the frost-defrost process and also due to the inappropriate quality of the lining of the carriageway, the runway has suffered important damages, located mostly in the areas between the tram rails.

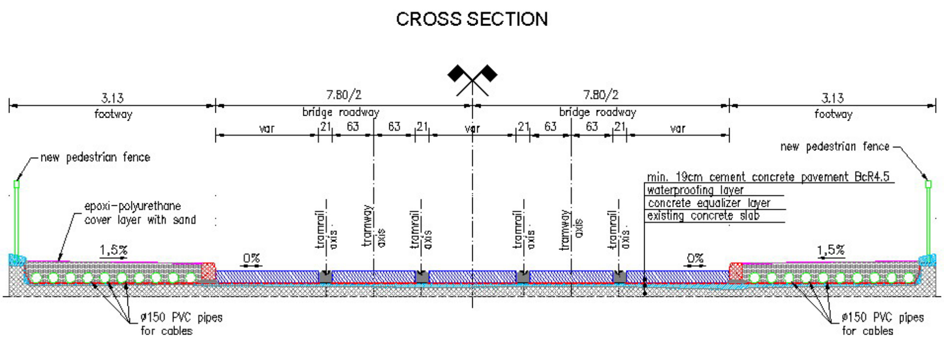

Fig.2 - Dacilor Bridge

The works considered to be necessary for the rehabilitation of the four bridges, are referring to the repair of the areas with exfoliated concrete, the entire restoration of the waterproofing, the footway and the runway, the disposal of new systems for the expansion joints, the fastening of the tram rail system using a modern and reliable solution, repairs at the pedestrian fence and the decorative elements if damaged by using a material which should regrant the structure the initial aspect (for the ancient bridges like Traian and Decebal), respectively its replacement at the other two bridges. For the protection of several areas of the bridge structure the choice of optimal qualitative solutions was aimed.

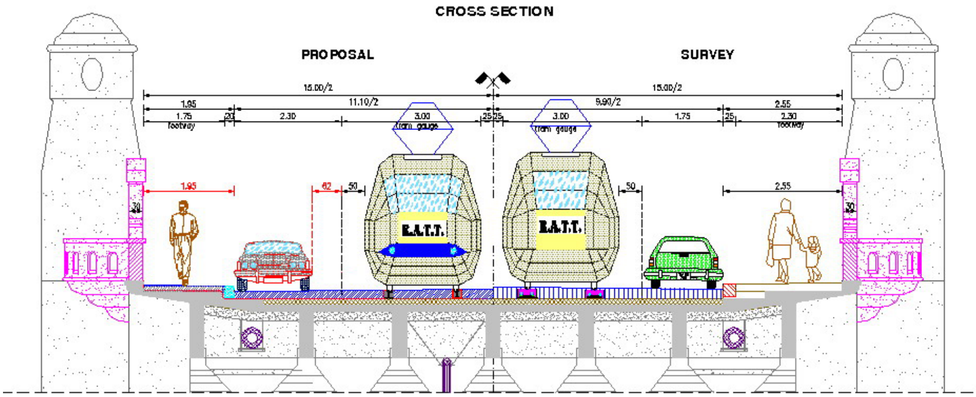

Fig.3 - Decebal Bridge

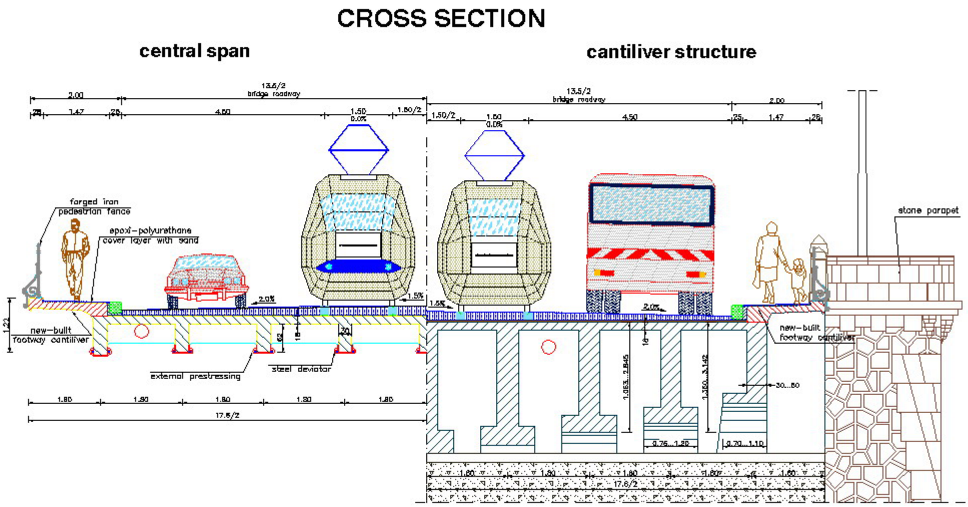

Fig.4 - Traian Bridge

The Traian Bridge has a limited bearing capacity due to the age as also because of the technical norms used at the time the structure was designed. These had stipulations regarding the loading values, which are much under those nowadays. The bridge is going to be consolidated by using external prestressing in order to satisfy the request of a higher loading class. Nevertheless the designs have to contribute to the maintenance of the architectural identity of the structure.

Photo 1 - Long view before of the starting of the works

The aim, concerning the discussed bridges, was the rearrangement of the gauge (runway and footway) in order to correspond to the actual traffic conditions and in perspective, as it can be seen in fig.1 to 4.

3. REHABILITATION SOLUTIONS FOR THE DACILOR BRIDGE

Due to the existence of an incomplete technical documentation and new elements, which arose after the works have started, the elaboration of some particular execution details related to the real situation were assessed.

The remove of the existing layers of the carriageway on the bridge has revealed a very irregular surface because of the deficiency at the placing of the precasted reinforced concrete plates which together with the steel girders form the mixt steel-concrete resistance structure (Photo 1, Photo 2, Fig. 5).

The execution of an equalization concrete layer in order to obtain a surface proper for the application of the waterproofing system but also for the execution of a curb in a vertical plane, an outcome of the redesigning of the longitudinal profile, became imperative (Photo 3).

When the transition plates were brought down, a reinforced concrete frame with a retaining wall, most probably a remainder from the structure of the old bridge, could be seen (Photo 4).

Photo 2 - Cleaned concrete plate

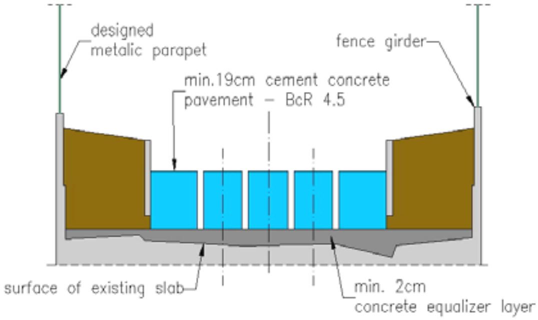

Fig. 5 - Transversal profile

The structure for the new retaining wall is based on this structure and will also support the designed transition plates (Photo 5).

Photo 3 - Equalization concrete layer

Photo 4 - Existing retaining wall

Photo 5 - New retaining wall

Photo 6 - The steel structure corrosion

Photo 7 - Anticorrosive treatment on the girder and at the end of the bridge

Because the corrosion attempt a high level (at only 15 years old), it was necessary to "clean" the steel structure with a performant and ecological system, based on water extra high pressure. Sika provides the multilayer anticorrosive treatment.

The solution for the superior layer on the footway consisting of using epoxi-polyurethan lining and also quartz-sand allows the obtainance of good waterproofing of the concrete in the footway and furthermore an asperity having an efficient non-slipping effect. It also permits the coloring of the pedestrian area.

Photo 8 - View before the works started

Photo 9 - Cleaned concrete plate

The transition between the superstructure of the bridge and the reinforced concrete plate of the tramway respectively the carriage way structure of the adjacent streets, is going to be realized by the means of special transition plates designed in correlation with the existing situation (bridge with cantilever and retaining wall).

4. REHABILITATION SOLUTIONS FOR THE MIHAI VITEAZU BRIDGE

The works for this bridge had the same starting conditions as the Dacilor Bridge (incomplete initial technical documentation etc). The Mihai Viteazu Bridge is realized with four traffic lanes on two parallel superstructures separated by a longitudinal joint.

In this situation the rehabilitation works were possible alternatively on each superstructure. The traffic was guided, in the mean time, on the other superstructure, which was not affected by the works (before, respectively after the rehabilitation) (Photo 8, Photo 9).

The remove of the existing layers revealed a very neat monolith plate, having only few geometrical imperfections. In contrast to the basic documentation, it has been found that, in the tram rail area the plate was made as a "small channel". As a consequence, the redesigning of the cross section and the renouncement of the paving concrete, was necessary and that made the choice of a much more versatile structure for the roadway possible.

Photo 10 - Damage in the pedestrian fence

Photo 11 - New fence girder

Due to the degradation of the concrete from the guard beam on large surfaces and in depth, respectively of the steel structure from the pedestrian fence, the replacement of the pedestrian fence and its girder was disposed (Photo 10 & 11).

Concerning this structure, the execution of the equalization concrete layer was necessary only for the correction of the grade line, respectively the realization of the cross slope for the discharge of water.

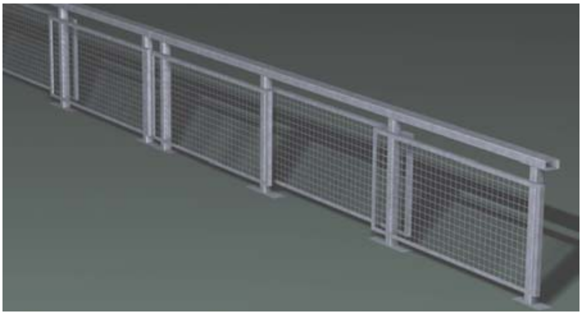

Photo 12 - Designed pedestrian fence

At the same time the fence girder is being repaired and the existing reinforcement is being completed, the new geometries of the element being taken in account.

The existing fence will be demolished so that the steel profiles remain enclosed in the new realized concrete girder. In that way they become connectors between the cantilever of the footway and the fence girder. The new designed fence is thought as a modulated steel structure, protected by zinc coating, presenting itself with a high transparency, considering the fact that it can respond entirely to the standard stresses (Photo 12).

5. THE TRAMWAY ON THE BRIDGE

In order to choose the optimal solution for the fastening of the tramway on the bridge (avoiding the perforation of the concrete plate of the bridge superstructure) a comparison was made between two systems: SIKA and ORTEC.

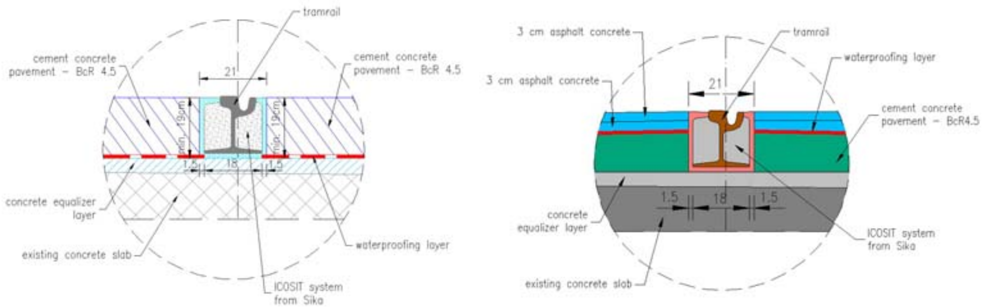

Fig. 6 – ICOSIT System on the Decebal Bridge and on the M. Vitezu Bridge

Photo 13 – ICOSIT System on the Decebal Bridge and on the M. Viteazu Bridge

As a conclusion, in order not to affect the concrete plate of the bridge superstructure by using fastening solutions which involves new perforation of the concrete plate (ORTEC system used on the street), the SIKA system was adopted. This system consists in fastening of the rail on the bridge by using ICOSIT KC 340/45 (Fig. 6).

Nowadays the rehabilitation works of the two- presented bridges are in process aiming the fastening of the rail by using the ICOSIT system, as national premiere (Photo 13).

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Railway Dislevelments Influence on the Dynamic Response of Bridges

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Abstract

The main purpose of this paper is to estimate the influence of railway dislevelments on the dynamic response of railway bridge structures subject to the load of a passing vehicle. The study covers six types of bridge structures: four steel structures and two prestressed concrete ones. A parameter study is performed to determine the dynamic amplification factor values and their variability as a function of railway dislevelments characteristics: amplitude and length. A special computer programme developed by the authors is used in the respective parameter study.

1. INTRODUCTION

The purpose of this study is to determine the influence of the railway dislevelments on the dynamic response of railway bridge structures, under the action of a passing vehicle. A parameter study has been performed, determining the values of the dynamic amplification factor and its variation depending on the characteristics of the railway dislevelmens (amplitude and length). The adopted values for these two characteristics correspond to those accepted by the Romanian Regulations [5, 6] in force. The parameters’ calculation is performed by means of a specific computer programme, elaborated by the authors.

Six case studies are considered, representing existing railway bridge structures:

(i) four steel structures namely deck with plate girder web bottom way, of 21.0 m span; deck with plate girder web, top way, of 30.0 m span; deck of truss girder, bottom way, of 21.0 m span; deck of truss girder, top way of 33.08 m span;

(ii) two structures with deck of precast prestressed section girder with post – tensioned reinforcement, of 22.0 m span (4 girder in cross section) and respective 30.0 m span (2 girders in cross section).

2. THEORETICAL APPROACH

The hypotheses and the modelling of the ensemble structure / vehicle is specified in the figure 1, respectively:

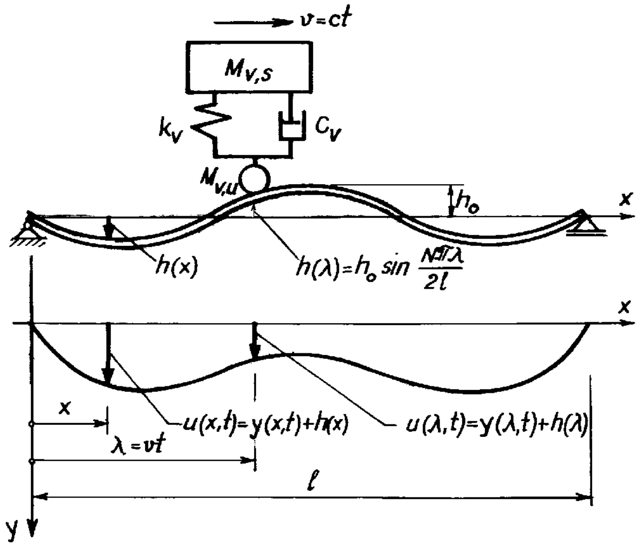


Figure 1. The modelling of the ensemble structure / vehicle

- The structure is modelled with dislevelment. The structural damping is introduced through the fraction of critical damping;
- The vehicle is modelled as a one degree of freedom dynamic system, consisting of a suspended masse, a non-suspended masse, a spring and a damper (elastic and damping characteristics).

The following simplifications are considered:

- The girder is simply supported ;
- The shape of dislevelments is sinusoidal;
- The vehicle is unique, modelled as a one degree of freedom dynamic system.
- The non-suspended mass is in permanent contact with the rolling surface.

Only the midspan section of the girder is considered in the analysis.

The used computer programme - OSIE1 - includes:

- INDOS1 input data programme;
- CALCOS1 analysis programme;
- DESOS1 drawing programme.

The central difference method is used in order to solve this system. The mathematical procedure is the step by step integration. The integration step is t/400 (where t is the necessary time for the vehicle movement on the beam of "l" span, with "v" velocity). A number of five velocities have been taken into account, namely v_0 = 5 km/h (at which the static deformation is obtained), v_1 = 60 km/h, v_2 = 90 km/h, v_3 = 120 km/h and v_4 = 160 km/h.

The dynamic amplification factor is obtained as a ratio between the maxim dynamic and the maxim static displacement values at the midspan section of the girder.

3. WORK METHODOLOGY

The study is performed in two stages, namely:
- *The first stage* of study consists in the determination of the natural vibration characteristics of the structures, mainly the type bending vibrations in vertical plan. The study is performed by modelling the structures with one and two - dimensional finite elements. For each structure many natural mode shapes are determined until the first three type bending mode shapes in vertical plan have been obtained.
Another study for determining the influence of the second and the third mode of vibration upon the dynamic response of the structures, under the vehicle action, was initially performed. It was concluded that they have a small contribution (5% is the highest value obtained only for the deck with truss girder, bottom way, of 21.0 m span at a speed of 160 km/h).
- *The second stage* of study consist in determining the dynamic amplification factor for:
    - the case of ensemble: structure with railway without dislevelment / vehicle, situation named "standard";
    - determination of the dynamic amplification factor for several situations of the railway with dislevelment, by varying the amplitude and the length of the sinusoids modelling the dislevelments / and the same vehicle as the standard situation. The standard vehicle is a railway engine with following features: the suspended mass = 10.0 t, the non–suspended mass = 2.8 t, the stiffness of the spring = 1740 tf/m, the damping of spring = 360.0 tf s/m.]

4. SITUATION OF STUDY CONCERNING THE VARIATION OF DISLEVELMENT

The Romanian Regulations in force requires a maximum value of the dislevelment amplitude of 10 mm with a junction of 12.0 m (the length of the sinusoid is 24.0 m). It means maximum one sinusoid (S=1), for the structures of 21.0 m span and

maximum one and half sinusoid (S=3/2) for the structures of 30.0 m respective 33.08 m.
It has been noticed as interesting the combined situation – number of sinusoid (S) with the value of their amplitude (h_0). Therefore a combination of the number of sinusoids (S=1/2, S=1, S=3/2, S=2) with values of the amplitudes varying around the maximal value required by the standard of 10 mm for railway (h_0 = 2.5, 5, 10, 15 and 20 mm) is proposed. In all cases the vehicle was the standard vehicle. The velocities for the vehicle movement were of 60,90,120 and 160 km/h.
As a result, the study comprised a number of 80 variants of combinations – h_0, S, v – for each structure, respective a total of 480 situations of analysis for six structures.

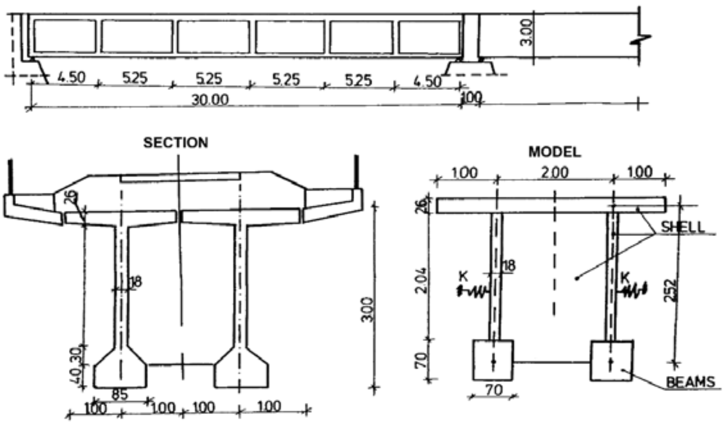
5. CASE STUDIES

In figure 2 is presented the deck of prestressed precast concrete of 30.0 m span and two girders in cross section = the general scheme of calculation, the discretized model, the forms 1, 2, 3 of type bending vibration in vertical plan. From the first stage of study it have been kept the natural characteristics of vibration for the structures.
In table 1, there are showed the geometric and dynamic characteristics for the six decks, used in the second stage of study.

Table 1

Deck type	**l (m)**	**M ($tf.s^2/m$)**	**ω (s^{-1})**		
			I	**II**	**III**
Plain girder web – bottom way	21.0	7.187	42.43	143.1	224.3
Plain girder web – top way	30.0	14.679	30.94	130.8	209.6
Truss girder – bottom way	21.0	6.486	58.69	130.8	161.6
Truss girder – top way	33.08	13.802	28.04	80.5	123.1
Concrete – 4 girders in section	22.0	45.31	35.08	120.7	261.7
Concrete – 2 girders in section	30.0	51.23	32.88	99.68	212.2

The second stage of study is performed by means of the specific computer authors’ programme. In figure 3 is presented the way of processing the input data and the output data for the programme. It is chosen the deck of precast prestressed concrete of 30.0 m span for the case of the sinusoid of 10 mm height and of a number of 3/2 sinusoids on the span. The four drawings of the figure are according to the four velocities considered in the study. The graphic representations comprise: the displacements of the weight’s centre of the vehicle, the structure with the sinusoidal displacements, the dynamic and static displacements of the midspan section of the deck. The next showed values refer to some parameters of the study.


4.50
5.25
5.25
5.25
5.25
4.50
30.00
100
3.00
SECTION
MODEL
100
200
100
26
SHELL
18
K
K
2.04
252
70
BEAMS
300
40 30
85
100
100
100
100

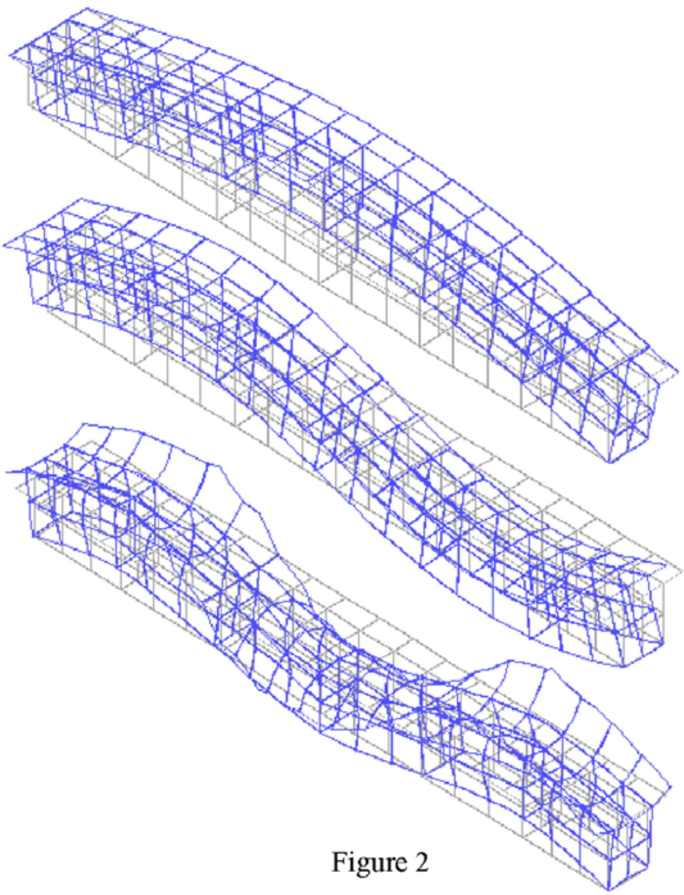


Figure 2

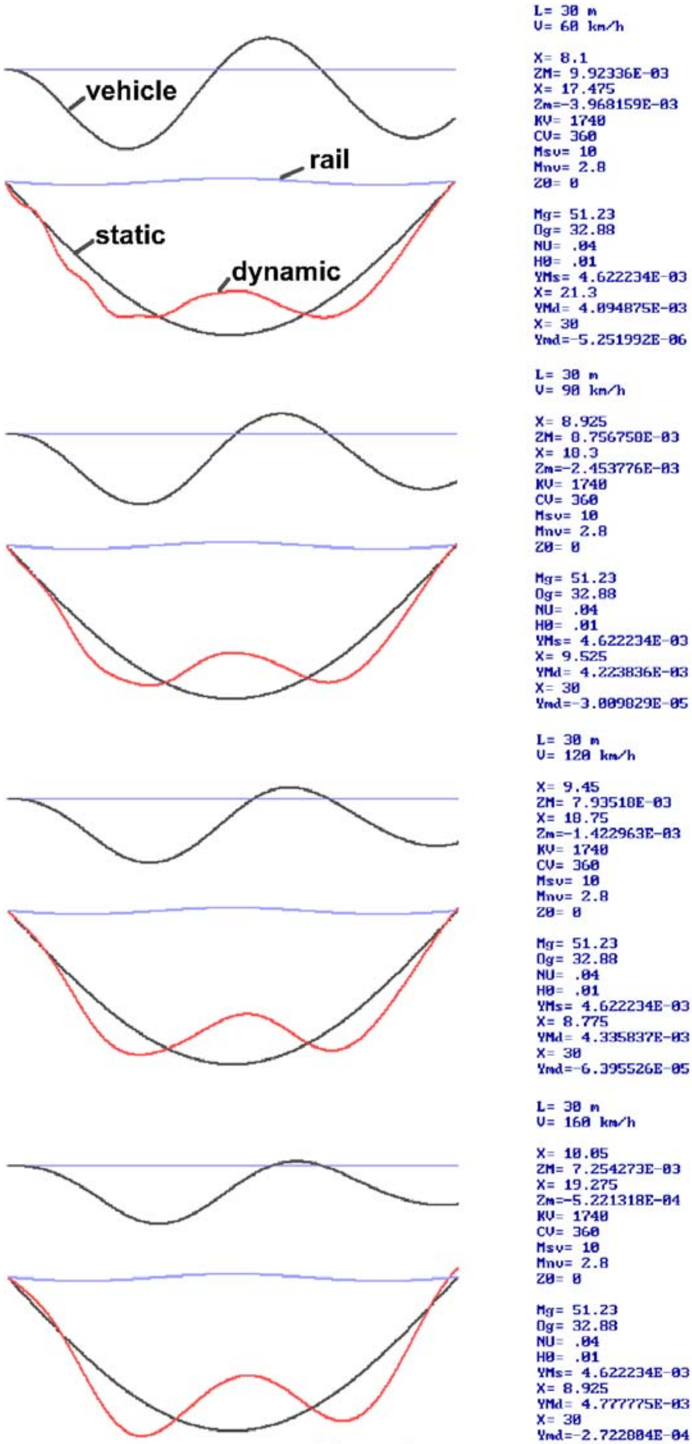

vehicle
rail
static
dynamic
L= 30 m
V= 60 km/h
X= 8.1
ZM= 9.92336E-03
X= 17.475
Zm=-3.968159E-03
KV= 1740
CV= 360
Msv= 10
Mnv= 2.8
Z0= 0
Mg= 51.23
Qg= 32.88
NU= .04
H0= .01
YMs= 4.622234E-03
X= 21.3
YMd= 4.094875E-03
X= 30
Ymd=-5.251992E-06
L= 30 m
V= 90 km/h
X= 8.925
ZM= 8.756750E-03
X= 18.3
Zm=-2.453776E-03
KV= 1740
CV= 360
Msv= 10
Mnv= 2.8
Z0= 0
Mg= 51.23
Qg= 32.88
NU= .04
H0= .01
YMs= 4.622234E-03
X= 9.525
YMd= 4.223836E-03
X= 30
Ymd=-3.009829E-05
L= 30 m
V= 120 km/h
X= 9.45
ZM= 7.93510E-03
X= 18.75
Zm=-1.422963E-03
KV= 1740
CV= 360
Msv= 10
Mnv= 2.8
Z0= 0
Mg= 51.23
Qg= 32.88
NU= .04
H0= .01
YMs= 4.622234E-03
X= 8.775
YMd= 4.335837E-03
X= 30
Ymd=-6.395526E-05
L= 30 m
V= 160 km/h
X= 10.05
ZM= 7.254273E-03
X= 19.275
Zm=-5.221318E-04
KV= 1740
CV= 360
Msv= 10
Mnv= 2.8
Z0= 0
Mg= 51.23
Qg= 32.88
NU= .04
H0= .01
YMs= 4.622234E-03
X= 8.925
YMd= 4.777775E-03
X= 30
Ymd=-2.722004E-04

Figure 3

Important observation: In the graphic representations of the values of dynamic *amplification factor a deformed scale is* adopted, [(the values of dynamic amplification factor – 1) x 1000]. Therefor, some *values from the graphic are below "1"*.

The values of the dynamic amplification factors obtained for the variation of the three parameters S, h_0, v are systematized in figures: 4 (a, b, c, d) - deck of plain girder web, bottom way of 21.0 m span (4a – S=1/2, 4b – S=1, 4c – S=3/2, 4d – S=2); 5 (a, b, c, d) - deck of plate girder web, top way of 30.0 m span; 6 (a, b, c, d) - deck of truss girder, bottom way of 21.0 m span; 7 (a, b, c, d) - deck of truss girder, top way of 33.08 m span; 8 (a, b, c, d) the deck of precast prestressed girder of 22.0 m; 9 (a, b, c, d) – the deck with precast prestressed girder of 30.0 m.

For the case in which the amplitude of the dislevelments "h_0" is kept invariably, namely at the maximum value admitted by the Romanian Regulation [5] (10 mm) and the others two parameters (S and v) are varying, the graphic representations are showed in the figures 4e, 5e, 6e, 7e, 8e, 9e.

In the figures 4f, 5f, 6f, 7f, 8f, 9f are represented the values of the dynamic amplification factors for the next situation: the maximum number of sinusoids required by the Romanian Regulations [5] depending on the span of each deck, the standard situation (railway without dislevelments), the value of the maximum dynamic amplification factor admitted by the Romanian Regulations [6] for each deck, for obtaining some comparisons.

6. COMMENTS. CONCLUSIONS

For steel structures (figures 4, 5, 6, 7), the following are to be noted:

1. The general shape of the graphs which represents the values of the dynamic amplification factor with the same number of sinusoids is similarly for all the structures.
2. The values of the dynamic amplification factor increase at the same time with the dislevelment amplitude rising (h_0).
3. The variation of the number of sinusoids (S) leads to an random behaviour , i.e.:
   - The most favourable situation is for the case S=3/2 (which means exceeding the maxim number of sinusoids admitted by norms with almost 3/4 sinusoid for the decks of 21.0 m span and at the most the maxim number of sinusoids admitted by the Romanian Regulations for the decks of 30.0m span), figures 4c, 5c, 6c, 7c.
   - The most unfavourable situation is that one for S=2; a sudden increase of the dynamic amplification factor at the velocity of 120 km/h as a phenomenon of punctual resonance.

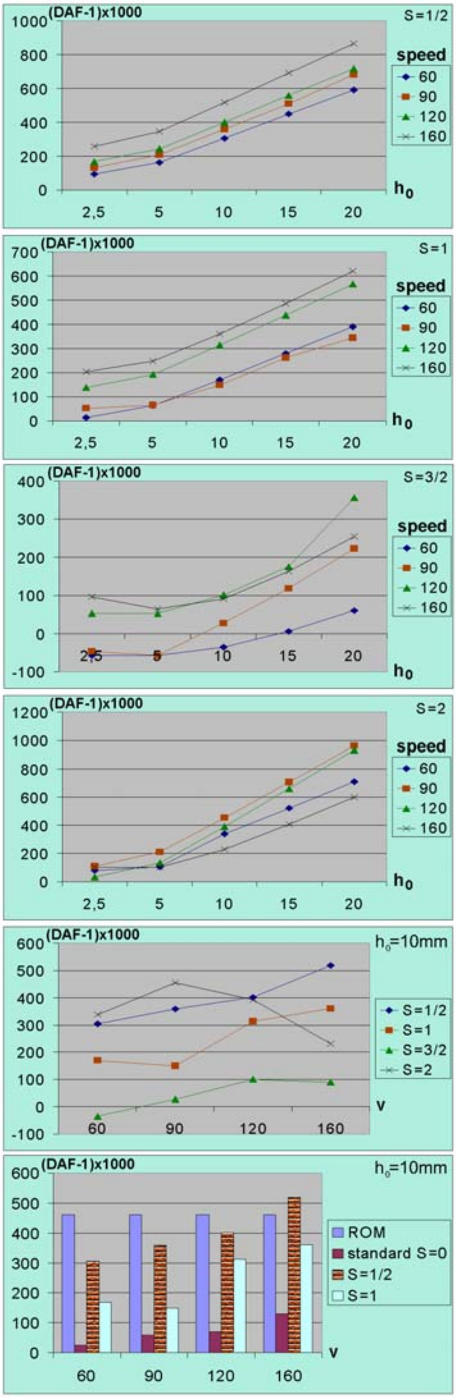


Figure 4. Plain girder web (21.0 m)

a.

b.

c.

d.

e.

f.

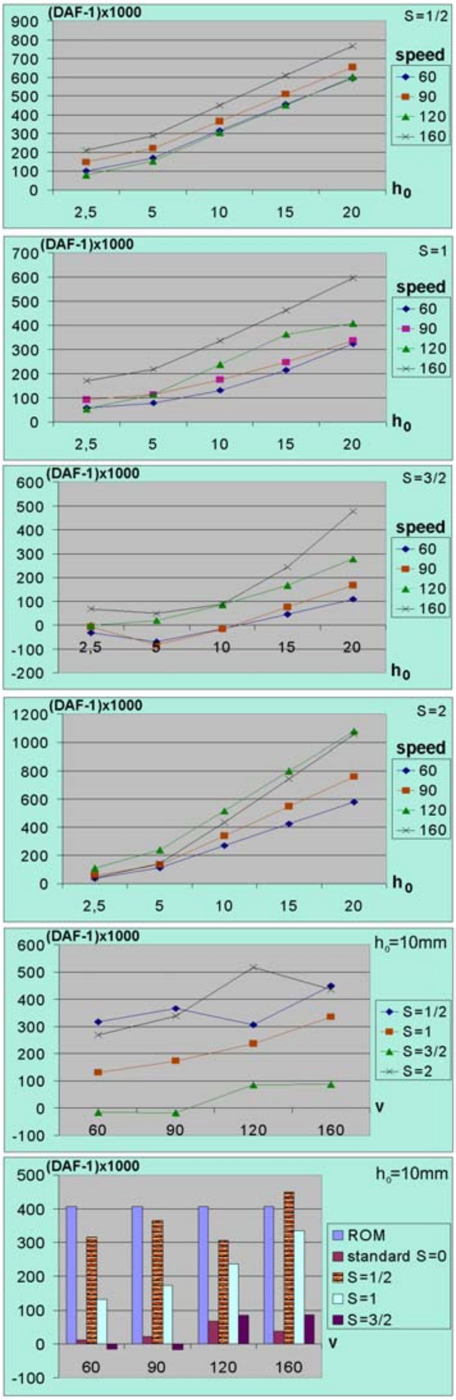
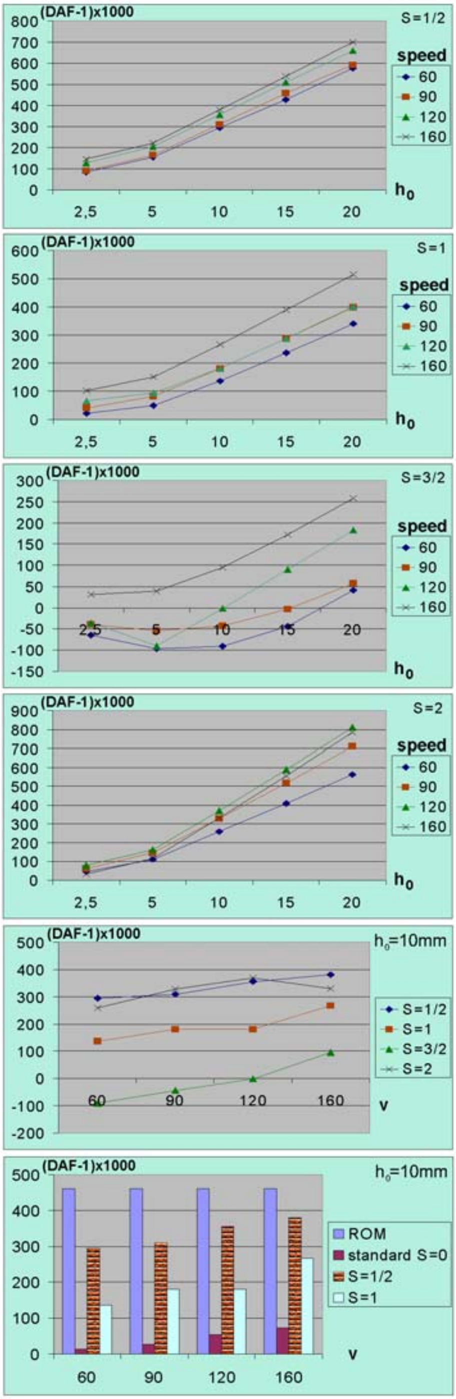


Figure 5. Plain girder web (30.0 m)



a.
b.
c.
d.
e.
f.

Figure 6. Truss girder (21.0 m)

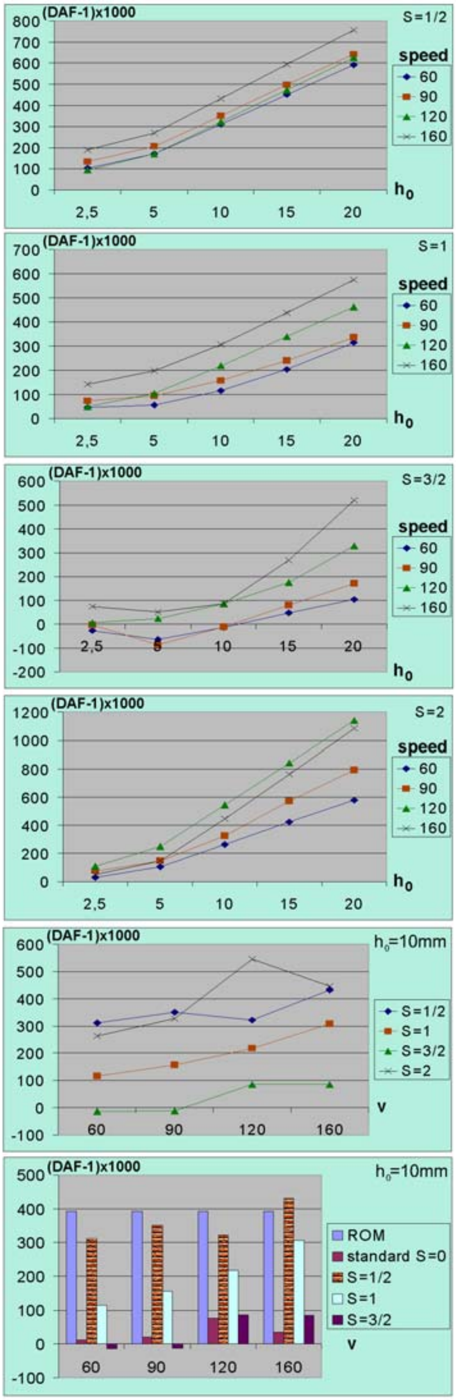


Figure 7. Truss girder (33.08 m)

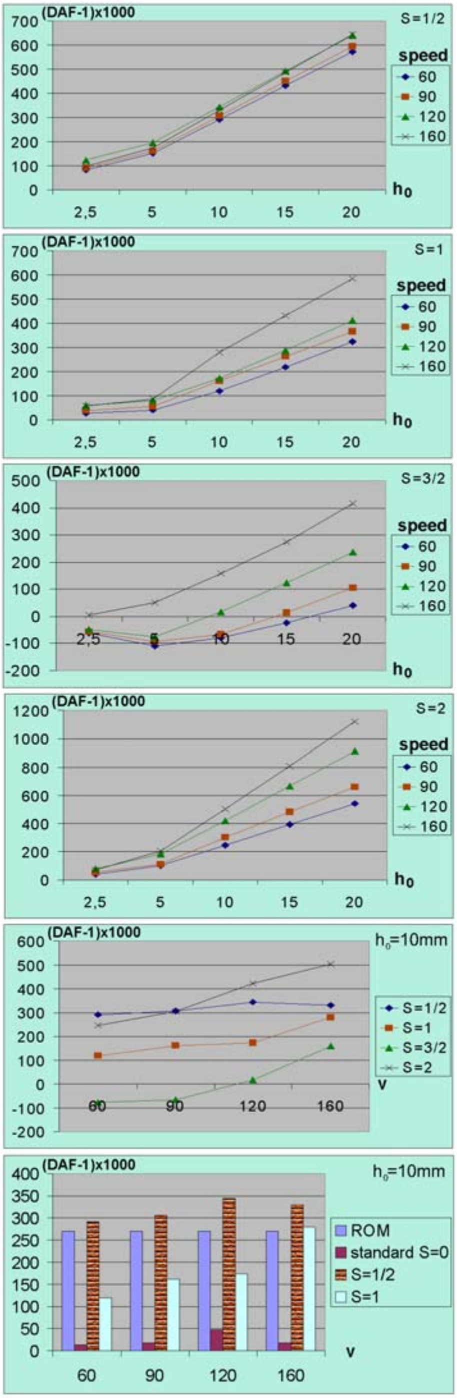


Figure 8. Concrete (22.0 m)

a.

b.

c.

d.

e.

f.

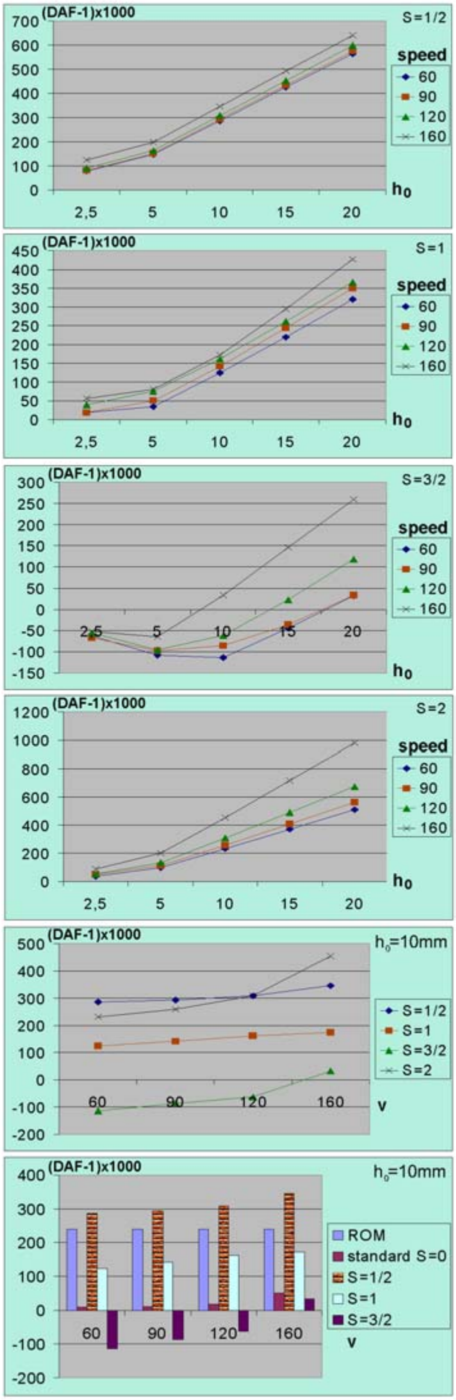


Figure 9. Concrete (30.0 m)

4. If is to be remained below the maxim number of sinusoids admitted by the Romanian Regulations – related to the span of each deck (figures 4e, 5e, 6e, 7e) – then the greatest values of the dynamic amplification factor are noticed for the case S=1/2 (a semi-sinusoid which overlaps under the static deformation of the structure).
5. From the graphs 4d, 5d, 6d, 7d it results that:
   - All the combinations S, h_0, v that are below the values admitted by the technical regulations in force (design speed = 120 km/h) leads to values of the dynamic amplification factor smaller than those allowed by the loads Regulations.
   - The case of speed of 160 km/h, (speed which is greater than the design speed), h_0=10 mm, S=1/2 leads to values of the dynamic amplification factor which exceed the maxim allowed value (excepting the deck of the truss girder of 21,0 m span).

For the concrete structures (figures 8, 9) it is to be notice that:
1. The influence of dilevelments is unfavourable in most of the combination of S, h_0, v parameter cases.
2. The values of dynamic amplification factor increase at the same time with the speed values.
3. The values of the dynamic amplification factor are generally rising at the same time with increasing the amplitude of the dislevelment.
4. The values of the dynamic amplification factor are decreasing at the same time with the deck length increasing with approximately 7%.
5. The greatest values of the dynamic amplification factor are obtained for the case in which the maxim amplitude allowed by the Romanian Regulations for the dislevelments (h_0=10 mm) is to be combined with a dislevelment length of one semi-sinusoid (S=1/2) on the whole span, a shape of the dislevelment which overlaps to the static deformed shape of the deck.
   - For this case the given value by the Romanian Regulations in force for the dynamic amplification factor is exceeded with about 6-8%.
   - The most unfavourable response way differs from one structure to another mainly depending on the speed. Thus, the maxim value of the dynamic amplification factor for the structure of 22.0 m is at 120 km/h and for the structure of 30.0 m span at a velocity of 160 km/h.

It is to be noticed that the dumping provided by the ballast prism hasn't been taken into account.

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8. Contract 423 / 1996 MEC Tema A20 / 2001 Influența denivelărilor căii asupra răspunsului dinamic al structurilor de poduri de cale ferata" Faza 1: Cale ferată pentru viteze mari – reabilitare poduri Tronson Bucureşti – Ploieşti

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Considerations Concerning the Environmental Project of Quality Management for Reabilitations Works for a Railway Bridge Case Study

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Abstract

The railways and road rehabilitation are recommended having in view the major changes in traffic's composition and, the development of security in transportations, generally. Thus, by a national program financed by EBRD, in Romania is developing an extensive program to rehabilitate the main national railways and roads.

Thus, for the part of European corridor between Bucharest- Campina were realized the rehabilitation of embankments (Ploieşti Triaj- West Ploieşti, Station West Ploieşti, West Ploieşti – Buda, Buda Station, Buda-Floreşti-Prahova, Station Floreşti-Prahova, Floreşti Prahova Câmpina) and bridges situated on route.

Some of bridges were built on a new route, like the bridge situated at km 45+075 over Prahova River, in order to ensure the safety in transportation at a velocity of 160 km/h.

The paper presents specifically steps required to ensure the environmental protection enclosed in bridge km 45+075 Bucharest – Câmpina, rehabilitation's project, on objects and physical stages.

Keywords: rehabilitation, environmental protection, pollutants

1. INTRODUCTION

Prahova River is one part of ensemble works for railways' rehabilitation Bucharest Brasov, part of corridor IV Pan European, for transportation by trains with a maximum velocity of 160 km/h.

The introduction of trains' circulation with a maximum velocity of 160 km/h imposed some modifications for the initial route as a fact of bridge km 45+703 rehabilitation.

In order to avoid the none wished effects of medium aggression as results of constructional works, was necessary to take an ensemble of measurements to protect the environment in face of constructional works’ impact.

2. CONSTRUCTIONAL WORKS

The location of river over Prahova River is shown in fig.1.

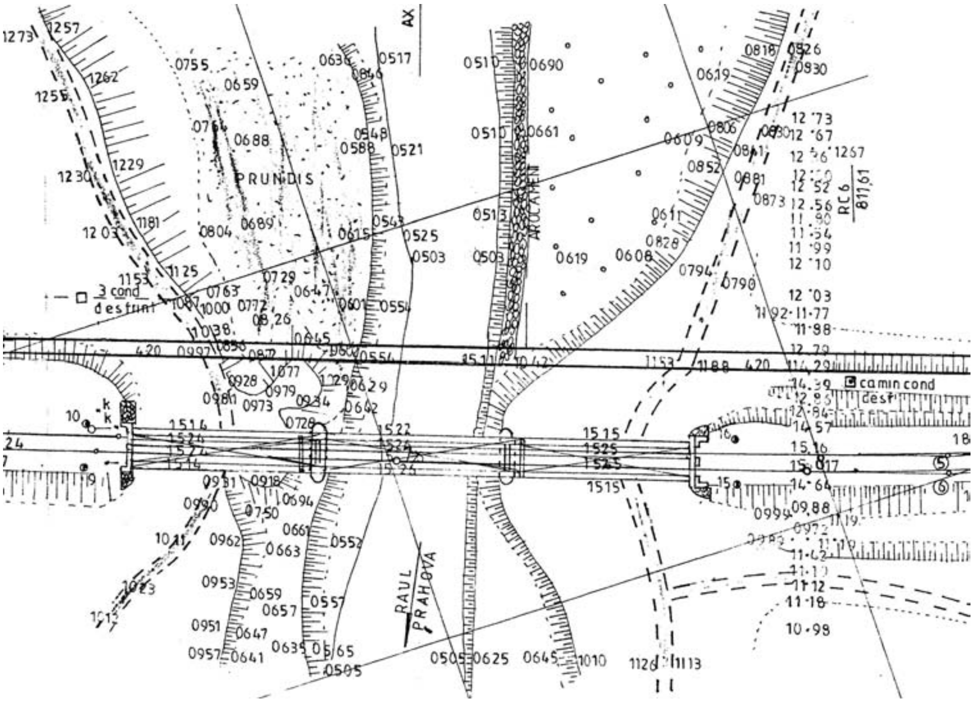

Figure 1. Location of Railway Bridge km 45+703

Embankment’s constructional works consisted in:
- The arrangement of technological roads along the railway;
- The cleanliness of right of way from grass and herbs;
- The evacuation of supplementary vegetal soil
- Digging till the quota on transversal profiles;
- Spreading of geo-textile material ;
- Finishing the soil fillings;
- Reinforcement of embankments;
- The execution of dripping out the water and of the stony ditches.

The protecting works consisted in:

- The accuracy of major river bed;
- The protection of shares with back fills by raw stone;
- The protection of river bed's bottom with raw stone mattresses.

The superstructure was realized by metallic structures for double way, with lattice works beams, down way and concrete cuvee in order to sustain the railway on ballast.

Foto 1. Ploiesti Pilon

The infrastructure was realized by using indirect foundations type concrete columns. The photos 1, 2 show some aspects during the constructional works.

3. ENVIRONMENTAL PROTECTION WORKS

The environmental protection works were executed in the same time with the constructional works, after the common program for quality management in rehabilitation works.

It can be mentioned the following specific works for environmental protection:

- Cleaning the terrain from strange corps (vegetal wastes) on a surface of 1000 m^2, in bridge zone and embankments;

- Manual mobilization of soil on a surface of 500 m^2, in order to ensure the plugging with vegetal layer;

- Dressing the embankments with natural soil on a surface of 400 m^2;

- Disaffecting of existing line (mechanically or manually after case).

Foto 2. Bridge km 45+703

It can be mentioned that soil resulted from digging was of 125,000 m^3 and was stored by quality in storehouses at km 45+500, respectively km 45+800 and loan hollows existing along the railway.

It can be shown the steps during the arrangement of soil storehouse:

- Soil's transportation to platform in the moment of battlefield works' advancing and after sorting necessary materials;

- In the case of platform realized in prolongation to railway in embankment, the superior level of soil in storehouse after final leveling was 50 cm lower than the level of embankment;

- The storehouse near the railway was realized in order to permit the water's drainage (pants are 3...5% at superior part and 1:1,5...1/3 at embankment;

- The soil was stored in layers of 30...40 cm thickness and compacted by 4...6 passing of a compressor cylinder after STAS 3197-1.2/90 and 7582/91 rules.

Table 1

Current number	Works to be controlled and verified	Wrote Documents at Control PV – Minute PVRC – Minute for Qualitative Reception; PVT – Minute for leveling up CRM – Note book for evidences and materials' reception	Who draws and signs reception's documents: I – Inspection in Constructions B-beneficiary E- constructor P - designer
0	1	2	3
I	**EMBANQMENTS**		
1.1	Leveling up the soil's Storehouse	PVT	B, E
1.1a	Verifying the busy surface	PV	B, E
1.1b	Verifying levels and pants indicated in plans	PVRC	B, E, P
1.1c	Reception of soil quality for coverings	PV	B, E
1.3	Verifying the quality of sowings and plantations	PV	B, E
II	**CONSOLIDATIONS**		
2.1	Leveling up the arranged surfaces	PVT	B, E
2.2	Verifying the busy surface	PV	B, E
2.3	Verifying levels and pants indicated in plans	PVRC	B, E, P
2.4	Reception of soil quality for coverings	PV	B, E
2.5	Verifying the quality of sowings and plantations	PV	B, E
III	**BRIDGES‘ PROTECTIONS**		
3.1	Leveling arranged surface	PVT	B, E
3.2	Leveling up the soil's Storehouse	PV	B, E
3.3	Verifying the busy surface	PVRC	B, E, P
3.4	Verifying levels and pants indicated in plans	PV	B, E
3.5	Reception of soil quality for coverings	PV	B, E
3.6	Verifying the quality of sowings and plantations	PV	B, E
3.7	Disaffecting the technological road	PV	B, E

Notes:

The locations of soil storehouses were determined in order do not block under snow the transportation on railway.

The storehouses will not compromise the stability of slopes. The soil stored is dusty clay with fine particles ($ø \leq 0.06$ mm) greater than 90%.

- The uniform spreading of vegetal soil over the soil storehouses on a surface of 500 m^2 had conducted the following operations: digging the soil in store house, loading, transportation of vegetal soil, scattering the natural soil, compacting the filling, etc.

Disaffecting the technological road of 110m length, 3m width, 45 cm thickness of ballast was done after the end of constructional works at the new bridge. The operations at disaffecting were: mechanical removing of ballast, loading, transportation at the established stores, transportation of vegetal soil, sowings and vegetal plantations.

4. POLLUTION SOURCES

Pollutants’ Emission in air

An important source of air pollution in the area of site is the impact of particles’ emission thanks the stone sorting and sieving processes. The higher values of particles’ concentration resulted in sorting process (unloading, sorting, loading, erosion) are in the area where the activity is developing and till the distance of 100 m of way’s axis. Another source is done by internal traffic of equipment.

Pollutants’ Emission in water

The intervention works in river bed and in the next vicinity generate and increasing of river’s water turbulence. The rehabilitation and columns foundations, the embankments’ protections need attention in order to reduce at minimum value of material loss (cements, dust) which can generate a local alkalinity for water.

The proposed solution for Prahova River was the storing of all constructional materials on locations which cannot permit their drawing into the river.

The used from equipments were recovered and gave up on strict evidence, to Petrom stations.

Noise Sources

Along constructional works it was mentioned a „work corridor" as area potentially sonorous polluted. The superior limit of noise was of 65dB_A for the sonorous pressure level continuous, equivalent.

Noise level in the work area was determined by used equipments (number, technical parameters and daily time of use).

Pollutants' Emission in soil

As potentially sources were the activities which generate wastes: rehabilitations works, site organization, current repairs of equipments.

An important category was those of unbinding the railway panels and metallic beams of old superstructure. After unbinding all these were stored in special stores.

5. QUALITY CONTROL

Constructional works and those of environment protection were developed after a common program established by specialty designer in respect to Romanian lows: Law 10/1995, Government Decisions 261/1994, 272/1994, 273/1994. In Table 1 are shown on physic stages, the phases of quality control.

6. CONCLUSIONS

The environmental project is part of technical project and executions details and both, with the quality control project compile the project of quality management.
The protection and remaking the natural conditions are obligatory requirements on man's intervention over nature and lead to preserve local ecosystems.
The rehabilitation of constructional works is doing having in view the social command and only in respect with environmental protection's law.

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European co-operation in COST 347 brings alt activities closer together

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ABSTRACT

For decades it has been an accepted fact that accelerated load testing (ALT) is one of the most important means of analyzing pavement behavior. In spite of large costs, high benefit-cost ratios can be expected from ALT research. However, these benefits are often obtained through national research programs that tailor the research to the specific interests of the country concerned. A Pan-European approach would result in a more robust outcome that had wider acceptance, and hence reduce the cost to the individual countries.

COST 347 was established in October 2000 under the European Commission with the objective to harmonize the scattered European ALT efforts. The harmonization is expected to lead to more efficient use of ALT research in combined efforts among the different countries participating in COST 347. This includes sharing of results, common testing methods, and co-operative research projects.

The results from COST 347 are very promising. A total of 17 European countries participate on a regular basis, so does the TRB Committee on Full Scale/Accelerated Pavement Testing, and close links exist to ALT centers in South Africa, Australia, and New Zealand. Among the direct outcomes of our work are a newsletter, and an e-mail based discussion forum. A co-operation between the HVS Nordic and Denmark regarding ALT research in semi-rigid pavements can also be attributed to COST 347.

Among the future developments COST 347 intends to work for the establishment of an international association for ALT.

INTRODUCTION

COST Action 347 *'Improvements in Pavement Research with Accelerated Load Testing'* was initiated in October 2000 under the European Commission's COST (CO-operation in the field of Scientific and Technical research) framework and will be completed in the autumn of 2004. It has the overall aim of developing a climate that will lead to a more harmonized approach to accelerated load testing of pavements in Europe. To carry out this task efficiently, the Action will also develop links with other National and International organizations whose interests are similar. This European and global co-operation will accelerate our understanding of pavement behavior and bring about improvements in pavement design, maintenance and materials.

Road transport is the most important mode of surface transport in Europe, and it is fundamental to its social and economic development. Approximately 500 billion euro is spent each year on European transport issues, and the majority of these are supported by the road infrastructure. Investment in road construction and maintenance is therefore very high and any improvements will have a significant effect on the overall efficiency of mobility in Europe. Accelerated load testing (ALT) or accelerated pavement testing (APT), as is often referred to in other parts of the world, is an important tool for analyzing the behavior of pavements. It gives the closest simulation of in-service pavements and it can be used to give a rapid indication of pavement performance that has a high level of credibility under traffic loading greater than that experienced in practice.

Throughout the world, there are now over 40 ALT facilities, with the most important facilities being located in Europe, Australia, New Zealand, South Africa and the United States *(1)*. The main European facilities are situated in Denmark, Finland, France, Germany, the Netherlands, Romania, Slovakia, Spain, Sweden, Switzerland and the United Kingdom. Many of the European facilities are under the management of organizations involved in the Forum of European National Highway Research Laboratories (FEHRL), which can influence and enhance the co-ordination between the approaches taken in different countries. These facilities represent major investments with high operating and installation costs that can be in excess of 5 million euro. Although they are expensive, the benefits generally outstrip the costs. However, research is often duplicated and results cannot be easily transferred between facilities. Consequently, there is a need to ensure that these facilities are utilized in the most efficient manner. The achievement of this goal will produce a climate for increased co-operation that will lead to:

- An increase in pan-European cost-sharing research ventures;
- Collaborative research ventures between the public and the private sector;
- A broadening in the base of non-owner organizations using ALT facilities;
- International co-operation with pavement research groups outside Europe.

The European Commission and National Road Administration and Highway Authorities throughout Europe are supporting the Action. The COST Action is highly appropriate as makes it possible for a large number of European countries to participate, and it also provides an opportunity for international observers. The obvious synergy between TRB Committee AFD40 (formerly A2B09) on Full Scale/APT and the COST Action and future sharing of information will benefit all.

ACCELERATED PAVEMENT TESTING IN EUROPE

The first European ALT facilities were the British National Physical Laboratory circular track (1912) and the Road Research Laboratory's (now TRL) Road Machine (1933). The oldest machine still in use was built in 1962 and Figure 1 shows that, since then, there has been continuous growth in Europe, with the bulk being commissioned in the period 1975-85. The majority the European facilities are fixed structures, where full or reduced-scale pavement sections are built in a concrete lined pit, and traffic is simulated by full or reduced-scale rolling wheels loaded by a device that is constrained to move along a path, usually linear or circular. The modern tendency has been to construct linear test tracks. Currently, Europe has ten full-scale test tracks (where full-scale pavement sections can be tested by full-scale rolling wheels), two installations where pulse loads are applied by hydraulic actuators and several small-scale facilities.

The fixed facilities are either outside or inside a building or climatic chamber. Only one of the ALT facilities; HVS-Nordic (the Heavy Vehicle Simulator operated by Sweden and Finland) is mobile and can be used to test existing in-service pavements. Europe has no purpose built roads test-tracks similar to Minnesota Mn/ROAD or WesTrack *(2)* situated in Nevada.

STRUCTURE AND SCOPE OF COST 347

COST 347 is a concerted European research Action that involves 17 European countries, with observers and corresponding members from Australia, New Zealand, South Africa and the USA. The technical requirements of COST Action 347 are described in Annex 1 of the Memorandum of Understanding (MoU), which can be accessed through www.pave-test.org *(3)*.

The main advantage of ALT is that it yields pavement performance information much faster than real-time load testing (RLT) or long-term pavement performance (LTPP) studies, in which experimental test pavements are built into the road network and observed under the influences of normal traffic and climate. This characteristic makes ALT facilities a key element in pavement research. Instrumented ALT experiments linked with laboratory studies of the constituent materials makes it possible to further develop analytical models intended for pavement design and maintenance. ALT facilities can be used to develop these models using closely controlled traffic loading, climate and materials over a reasonably short time scale. However these models then need to be verified with RLT sections under the more complex conditions experienced in service like material ageing and real climatic conditions. Examples of RLT are the long-term pavement performance studies active both in the USA and Europe. In Europe the LTPP studies were examined under COST Action 324 and the EC Fourth Framework Plan, PARIS (Performance Analysis of Road Infrastructure, 1999) *(4)*. These have already extracted useful information from some European ALT facilities and linked this information to the performance of in-service experimental pavements in Europe.

In addition to these projects, COST 347 will help complement several already completed COST Actions *(5)*. These include development of a new bituminous pavement design method (COST 333), weigh in motion of road vehicles (COST 323), effects of wide single and dual tires (COST 334) and reduction in road closures by improved road maintenance procedures (COST 343). COST 347 also exchanges knowledge and ideas (and shares people) with the FORMAT (Fully Optimised Road Maintenance) *(6)* and TREE (Transport Research Equipment in Europe) *(7)* projects, both organized under the EC Fifth Framework Plan.

The technical annex of the MoU recognizes that to achieve the objectives will require collecting and collating information from the ALT facilities in

Europe. Therefore, six work packages together with deliverables shown in Table 1 were defined. The six work packages are mutually complimentary. Each of the working groups consists of five to eight members, and each member will generally sit on two or three working groups. This and the regular reports to the management committee ensure good communications between the groups. In addition, the observers and corresponding members bring a wider perspective to the work.

The Action began in October 2000 and is due to finish in October 2004. At the time of writing some of the work packages are incomplete.

SCIENTIFIC PROGRAM

The first two work packages are concerned with collecting information regarding European ALT facilities as well as the results obtained from the experiments that have been carried out in them. Work package 3 addresses the important issues concerned with relating ALT results to the performance of in-service pavements. Work package 4 draws on the work of the first three groups to develop a common code of good practice for ALT to facilitate closer collaboration between different ALT teams within Europe and elsewhere. The future direction of ALT testing is addressed by work package 5 and the last working group is responsible for the continuous dissemination of the work throughout the Action.

INVENTORY OF EUROPEAN ALT FACILITIES

The first part of COST 347 involved developing databases for the main characteristics of the facilities, monitoring and measuring methods together with how data is collected and treated. It also examined how pavement condition is evaluated and the laboratory and analytical support required for ALT. This information was collected by questionnaires sent to 19 identified European facilities. The 19 test facilities all fulfilled two requirements:

- The pavement section could be either full or reduced-scale, but it should have several layers;
- Traffic loading could be simulated by means of rolling wheels, hydraulic actuators or other devices.

The inventory of ALT facilities is described in more detail in the paper given at this Conference by Mateos and Balay *(8)*, so only a few highlights will be provided here.

Comprehensive responses from 14 facilities were used to compile three databases regarding the characteristics of facilities, instrumentation and pavement condition evaluation, and from each of these a report was produced that represents the European state-of-the-art. Contained in these are detailed descriptions of each European ALT facility with illustrations and diagrams. The locations and a few statistics for the facilities are shown in Table 2.

The inventory concluded that realistic loading and pavement construction are considered to be of prime importance for ALT. Linear test tracks usually fulfill these requirements with the exception that tracking speed is generally much lower than traffic speeds, while circular test tracks usually fulfill all requirements. With pulse loading devices, a pulsed plate load is used to mimic a rolling wheel load. Realistic simulation for all aspects of the environmental has not been achieved in any type of facility. It should be remarked that none of the ALT facilities reviewed simulate all aspects of the real-life situation, but the client can have far more confidence in their results compared to those obtained from small-scale laboratory testing.

The study of instrumentation in ALT found that near surface stresses or strains are rarely monitored, which suggests that cracking and in-depth rutting are not being studied adequately. Studies limited to the region of the wheel path may diminish ability to fully understand performance mechanisms. Reproducibility of measurements is often poor, therefore replicate measurements are essential. Consideration should be given to maximizing the collection of information for a given experiment and not only adhering to a pre-defined test schedule.

A series of studies conducted at the Swiss Federal Institute of Technology at Lausanne (EPFL-LAVOC) examined seven different asphalt strain gauges installed in the Halle-Fosse ALT facility. The performance of the different sensors was studied under different conditions regarding temperature, tire pressure, etc., and laboratory responses were compared to responses from ALT. In addition measured and calculated ALT responses were compared using different theoretical response models. The studies found, not surprisingly, significant differences between the responses from the different instruments and that linearity and repeatability of the instruments were good for all sensors.

Information on methods of quantifying different types of distress revealed that during construction, layer thicknesses, materials densities and the bearing capacity of each layer were measured in the majority of ALT trials. Most facilities use the Falling Weight Deflectometer extensively, and visual monitoring, measurement of permanent deformation and investigation by excavation at the end of the trial are carried out in the majority of trials. The frequency of monitoring is higher at the beginning of a trial, but it also depends on the performance of the pavement.

Test termination criteria differ among the facilities. The most common criterion is permanent deformation but the value varies from 18 mm to 50 mm. The comment from LCPC of France is believed to be relevant: “It's important to have maximum pavement damage to help with the assessment of pavement durability”.

PREVIOUS AND CURRENT ALT RESEARCH

The second inventory of COST 347 concerned research conducted using ALT and it drew together results and developments achieved by ALT research programs to date. Materials research, pavement performance models, pavement design and socio-economic considerations were covered. The output describes the situation both within and outside Europe concerning the analysis of ALT results. Work included suggestions for the structure of a database for future research results from European ALT facilities.

The inventory of ALT research is described in more detail in the paper given at this Conference by Mateos and Balay *(8)*, so only a few conclusions will be presented here.

The main research topics and the technical results obtained from ALT have been entered into two databases (ALT tests and publications). Information concerning experiments and publications was obtained by questionnaire. The data collection on test programs was restricted to Europe, while the request for literature relating to ALT was more widely based and went to ALT owners in South Africa, New Zealand, Australia and the United States. The literature database, which can be accessed via www.pave-test.org, contains approximately 600 entries categorized in topics like materials, pavement performance, pavement design, maintenance, wheel load effects, test validation and climatic effects.

The inventory of research results concludes that a very large number of current or new pavement structures have been tested or compared in European ALT tests, and that there is a very large extent of topics, which have been and may be usefully tackled with ALT testing. Yet, some progresses may be done about very actual topics such as maintenance or environmental effects. It is found that one of the most interesting benefits ALT is to provide quick assessments and/or comparison of current or new techniques. Figure 2 shows, as an example, the relative frequency of the various research topics from 1978 to 2002. A detailed analysis indicates that trials involving environment aspects (wastes, reclaimed materials) seem to be more frequent during the past few years.

There has been a strong development of ALT research over the last 20 years. This could be due to a need for quick assessment of new and/or proprietary products and techniques for road construction. Another reason could be a need for a performance-related approach to pavement behavior, in order to allow such new techniques. Most of the tests are performed with a large part of public funding. Yet, a number of them are performed with industrial partnership, which result in various implementations, going from at least reporting and publication up to guidelines, standards, and innovation or industrial development. European ALT tests are generally national tests. Cost-benefit analysis of ALT tests is, unfortunately, not common use.

Comparison of ALT and Real-time Load Testing

We examined the appropriateness and benefits of ALT facilities in relation to testing experimental sections of pavement built into the road network (Real-time Load Testing). The parameters studied were economy, construction techniques, physical and environmental conditions, load levels, and degree of pavement deterioration.

The overall conclusion was that ALT and RLT are very different types of test and their relative roles need to be defined to ensure that each type is used to its full potential. A straightforward comparison would be unfair and misleading as the context in which they are used needs to be considered. Generally, the methods compliment one another in the manner illustrated in Figure 3.

The diversity of ALT facilities makes it impossible to draw universal conclusion that cover all facilities. However, the following general observations are made:

- The main characteristic of ALT facilities that differentiates them from RLT trials is that is possible to obtain data of much higher quality. All aspects of ALT can be controlled to a much higher degree. This makes it possible to examine the influence of selected variables, which makes ALT ideally suited to be a research tool and, for example, to provide feedback in the development of pavement response models or mechanistic pavement deterioration models.
- The main characteristic of RLT trials is that they can be used to assess long-term pavement performance under actual climatic and loading conditions. This performance takes into account factors such as long term ageing of asphalt and other long term trends that cannot be reproduced in ALT facilities. Deterioration mechanisms, such as cracking in asphalt, that are dependent on ageing cannot be easily investigated with ALT. ALT facilities are better suited to examine short-term deterioration

mechanisms, such as excessive rutting due to adverse hot weather conditions, increased rate of rutting that results from the use of wide base tires (super singles), freeze thaw effects, etc.

- Both ALT and RLT can be used to assess the relative benefits of a new material or structure by comparison with a control section constructed using conventional material or construction. With ALT, the comparison may be in terms of performance or the performance implied by the reduced stress or strain response at critical locations in the pavement structure. Whereas with RLT, practicalities concerned with specifications, material production and pavement construction practice can also be examined.

The main difference between ALT and RLT is in the method of accelerating performance. RLT deterioration proceeds at the normal rate but with ALT behavior is normally accelerated by using higher axle loads, thinner pavement structures, faster and more continuous loading or adverse climatic factors.

ALT and RLT are both used to carry out complex investigations into the behavior of pavement structures. Both types of test have their relative strengths and weaknesses. These strengths and weaknesses can only be discussed in relation to the objectives of any trial that is being undertaken. For some forms of testing the ALT may be considered to be the best choice and conversely there are instances when the best choice would be RLT. Currently there are no transfer functions to relate ALT to RLT results because of the complex and unknown interactions between the many structural, loading and environmental variables. To obtain a robust assessment of long term pavement performance within a realistic time-scale, the two test types have to be combined. ALT and RLT are both engineering tools that help bridge the gap between laboratory studies and in-service behavior.

COMMON CODE OF GOOD PRACTICE FOR THE APPLICATION OF ALT

Many pavement engineering problems that are tackled by ALT facilities could be solved more efficiently if there was a concerted course of action involving more than one ALT facility. Before this can happen, a common code of good practice needs to be established so that results can be readily exchanged. It must also be recognized that not all ALT facilities are suited to solve a particular problem, and co-operation is required to ensure that each ALT is used in the best possible way.

The core of the common code of good practice developed by COST 347 consists of a number of recommendations, which should aid the ALT user in conducting the experiment in the optimum way both regarding organization, the technical side, as well a the financial side. Recommendations have been developed for the following items:

- Planning of experiment;
- Test pavement;
- Construction testing;
- Loading;
- Climate;
- Data;
- Pavement condition evaluation;
- Pavement instrumentation;
- Supplemental laboratory testing;
- Operational safety, public hindrance and environmental protection;
- Staffing;
- Economy.

All the 1-2 page long recommendations contain a title (as above), an aim, an importance rating, a number of mandatory actions, a number of recommended actions, as well as a list of references. The style of the recommendations is very general, the intention being to establish a framework for good ALT work, but on the other hand giving room for local flexibility. The references, on the other hand, give selected practical examples to implementation of the specific item.

The code of good practice is intended to help current users develop habits that will improve the quality and interchangeability of their results. At the same time they should be able to find guidance regarding new developments of the facility. For new users the code is intended to provide a guideline for establishing a well-functioning ALT program.

FUTURE USE OF ALT

COST Action 347 will have an important impact on the nature of future ALT testing. For example, it will lay the foundations for a better climate for ALT research in that it will be easier to develop co-ordinated programs involving international collaboration. With this scenario in mind, we examined four different areas for the future direction and use of ALT. The first considers future research into alternative and new road construction materials; the second is an investigation of ALT techniques for monitoring pavement material response, climatic influence, as well as pavement condition evaluation. The third area deals with the ALT investigations for different types of maintenance work, including those applicable to utilities and buried structures, and the fourth area considers how to evaluate pavement environment related issues in ALT facilities. The deliverable provides a catalogue of new pavement research topics to be investigated in ALT facilities.

DISSEMINATION AND INTERNATIONAL COLLABORATION

Throughout the COST Action dissemination of the progress and results has been given a high priority, via web sites, newsletters, personal contacts, seminars and scientific articles. Professional links have been established with all European ALT research centers as well as a number of centers in North America, South Africa, Australia and New Zealand.

Information via the Internet

The internet has been recognized as an important media and the Action has a dedicated web site (www.pave-test.org) with regular updates of the progress of the Action. The site is a focus for those interested in the Action and it is also a portal through which other related sites can be accessed. The web site, the opening page of which is shown in Figure 4, provides the user with the latest information and results from COST 347 as well as from TRB Committee AFD40. Members of COST 347 can enter a password protected area with internal documents, which are often still under development. All visitors to the site may access information about and links to most ALT facilities in the world. A news and events list is also available. Finally, links offer to direct users to the internet pages of the European COST framework and TRB, respectively.

COST 347 produces a regular six-monthly newsletter, ALT-MATTERS. Every issue features an editorial, ALT related articles from around the world, as well a calendar with ALT related arrangements. The newsletter has been very well received and it is sent electronically to approximately 200 people in all parts of the world. The newsletter can be seen on the Action's web site, too. Papers and reports were produced at important stages of the Action and published in national and international journals and presented at appropriate conferences as well as being available subsequently on the web site.

An e-mail based discussion forum, the Pave-Test Mailing Group, is co-ordinated through COST 347 and has approximately 175 participants. Any subject regarding ALT can be sent to the host of the mailing group, which then forwards the contribution for discussion among the group members.

Seminars

COST 347 has arranged or participated in three major events, which all proved to be excellent forums for exchange of experience in ALT research among the participants.

An international seminar on ALT facilities was organized at the outset of the Action with invited guests and speakers from New Zealand, South Africa, USA and several European countries.

The Sixth International Conference on the Bearing Capacity of Roads and Airfields, Lisbon (June 2002) hosted a workshop on in situ stress-strain measurements with participation and contributions from COST 347.

The Ninth International Conference on Asphalt Pavements, Copenhagen (August 2002) hosted a dedicated workshop on accelerated testing. The workshop was organized jointly by COST 347 and TRB Committee AFD40. The workshop gave an overview of the current worldwide state-of-the-art of ALT and further included topics like benefits of ALT to pavement engineering, ALT as a standard tool, exchange of information between colleagues, and future ways of networking. The discussion at the workshop showed that ALT may very well be the method used by contractors to reduce their risks in projects under new contractual relationships or in projects where new materials are used. The debate also stressed the importance of directing the marketing efforts regarding ALT findings specifically to the different groups of receivers, e.g., practitioners, researchers and decision makers. The debate finally demonstrated a large wish for co-operation and collaboration in projects involving ALT, and as a result of the workshop the e-mail based discussion forum about ALT was established.

The Future beyond COST 347

When the Action ends in October 2004, a final report will be produced in the form of a 20-page report aimed at potential customers for ALT trials, which would be road owners and companies producing road construction and maintenance systems. The report will summarize the work of the Action, while the reports produced as part of the work would be available on CD for those wishing to delve into the technical details. The material will obviously also be available on the Action's web site.

Once COST 347 terminates its work at the end of 2004 there is no formal European platform for the co-operation in ALT. In the light of the activities in COST 347 over the last four years it would be a shame not to continue the activities under a new umbrella. We will therefore explore the possibilities of creating a worldwide ALT Association (or User Group), which will host a dedicated ALT web site, a discussion forum, produce a regular newsletter, and possibly arrange a regular ALT conference or symposium. The intention is to establish a body independent of geography, type of ALT facility, etc., and a place for people interested in ALT to meet and exchange ideas and experiences.

ACCOMPLISHMENTS OF COST 347

The economic dimension of the COST Action 347 is immense. European expenditure on the road infrastructure runs into many billions of euros and the associated costs to users through congestion and disruption due to road maintenance are substantial. ALT facilities are a key tool in the optimization of road materials and in developing superior models for use in pavement maintenance and design that will lead to more durable and better maintained roads. An Action improving the efficiency of ALT research by developing a climate in which pan-European and International co-operation can take place will result in significant cost savings over the European road network.

COST 347 has accomplished its goals regarding the improved and increased European and International co-operation in ALT. Some twenty countries have worked together in the Action; people have made new professional relationships which are expected to be useful in the future. ALT experts have worked with less experienced colleagues and have hence helped disseminate knowledge about ALT, which in turn is of interest to the European Community at large. Europe has seen a growth in co-operative ALT projects, with the Swedish-Finnish Heavy Vehicle Simulator being used for projects in Poland and Denmark.

A platform for ALT research now exists including an overview of the existing ALT facilities in Europe, guidelines for their use, and ideas for projects. Furthermore, as the national experts know each other personally the scene is now set for real pan-European and International co-operation in ALT.

ALT facility owners now realize that they need to justify the relatively high expenses for ALT research in the language that their clients speak. Hence, any future proposal for ALT testing should be accompanied by not only an estimate of the expenses related to the testing, but just as importantly by an estimate of the benefits, which can be gained by the client by sponsoring the experiment.

ACKNOWLEDGEMENTS

On behalf of the COST Action 347 Management Committee, the authors would like to thank the European Commission for its support for the Action and also the assistance of friends and organizations worldwide in providing information necessary for the success of the Action.

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List of tables and figures

TABLE 1 Program and structure of COST Action 347

Work Package	Title	Task	Title	Deliverable
1.	Inventory	1.1	ALT characteristics	Database, SWOT analysis (strengths, weaknesses, opportunities and threats) and report.
		1.2	Instrumentation	
		1.3	Condition evaluation	
		1.4	Laboratory testing	
2.	Previous and current ALT research	2.1	Materials research	ALT research results, database and structure for ALT research database.
		2.2	Performance models	
		2.3	Pavement design	
		2.4	Socio-economic aspects	
3.	ALT versus RLT comparison	3.1	Economic considerations	SWOT analysis and report
		3.2	Construction techniques	
		3.3	Environmental conditions	
		3.4	Loading conditions	
		3.5	Pavement deterioration	
4.	Common code of good practice for ALT	4.1	Guidelines for existing facilities	Report
		4.2	Guidelines for future facilities	
5.	Future use of ALT	5.1	Alternative and new materials	Report
		5.2	New maintenance techniques	
		5.3	Maintenance works	
		5.4	Environmental issues	
6.	Dissemination	6.1	Exploitation of results	Seminars, workshops, papers and final report
		6.2	Final report	

TABLE 2 European accelerated pavement test facilities

Id	Location	Type of facility	Max tracking speed (km/h)	Loads per month (x 1000)
1	Lyngby, Denmark	Linear	25	50
2	Oulu, Finland	Linear (small-scale)	5	430
3	Delft, Netherlands	Linear	20	400
4	Sweden / Finland	Linear	12	600
5	Lausanne, Switzerland	Linear	12	400
6	Crowthorne, UK	Linear	20	500
7	Nottingham, UK	Linear (small-scale)	12	111
8	Madrid, Spain	Linear/Circular	60	100
9	Nantes, France	Circular	100	500
10	Iasi, Romania	Circular	40	51
11	Bratislava, Slovakia	Circular	50	170
12	Zürich, Switzerland	Circular	80	83
13	Bergisch Gladbach, Germany	Pulse Loading	-	1000
14	Dresden, Germany	Pulse Loading	-	12000

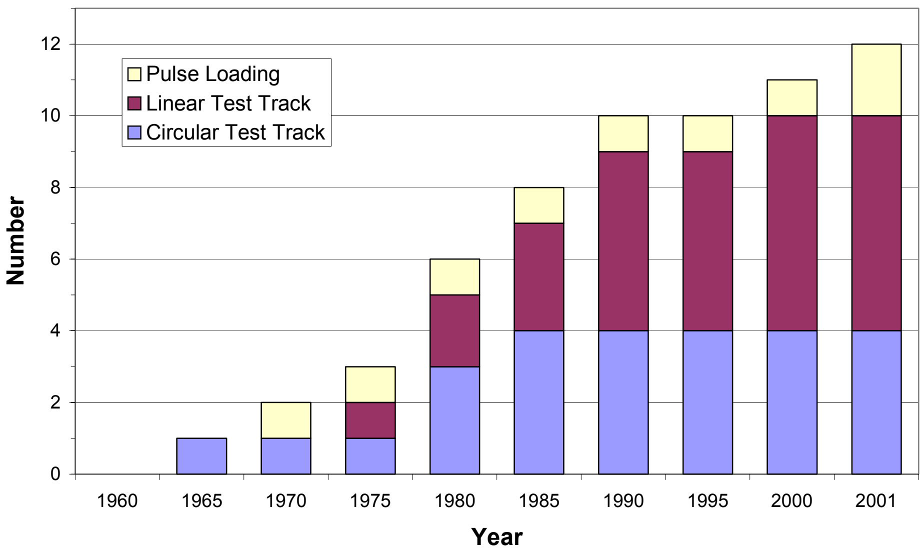

FIGURE 1 ALT facilities operating in Europe.

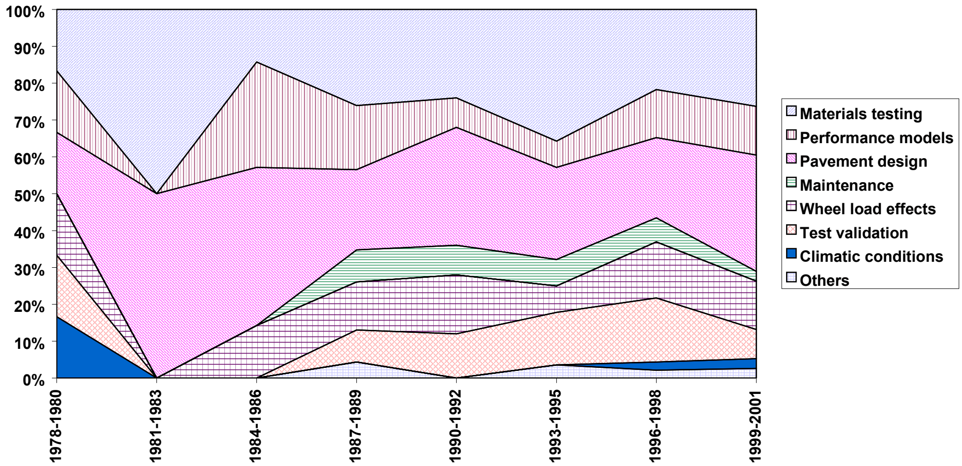

FIGURE 2 Breakdown of ALT research topics over time.

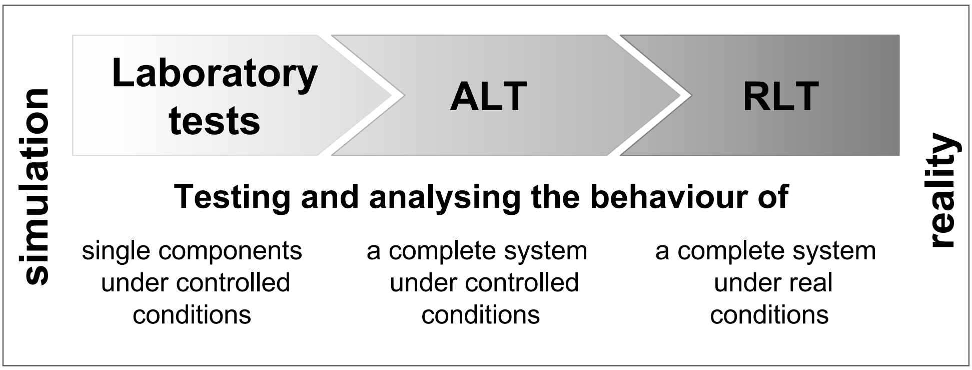

FIGURE 3 The relative roles of methods of testing.

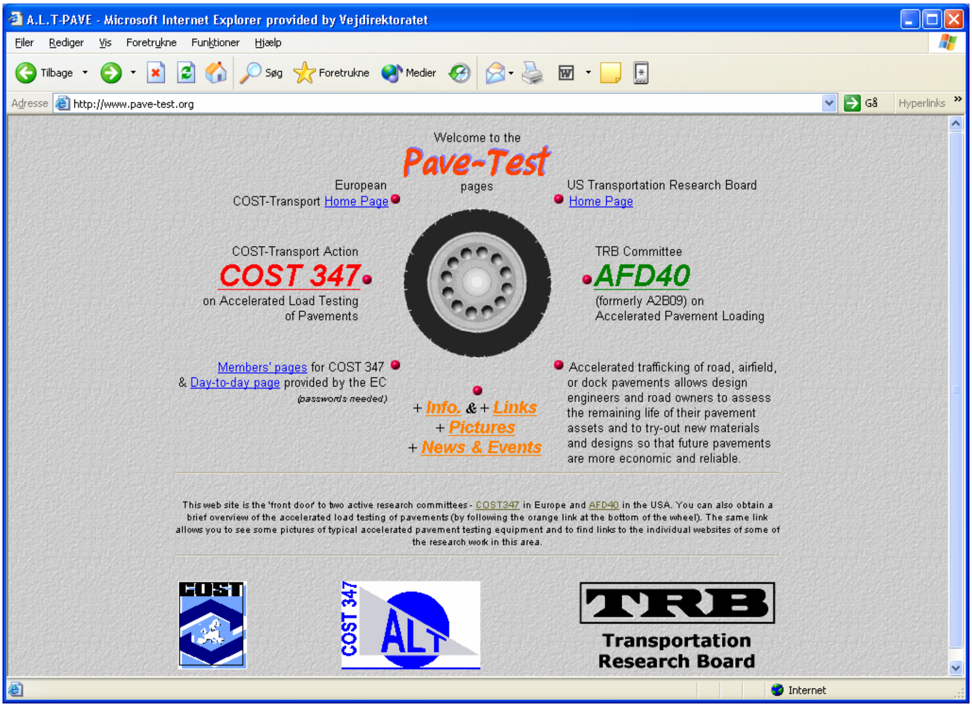

FIGURE 4 The opening screen of the ALT web portal www.pave-test.org.

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New non-destructive diagnostic method of bridges

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Abstract

Currently, on the base of foreign experience the Bridge Management System of the Czech Republic is created and progressively implemented. In connection with this process the requirements for non-destructive testing realized directly during the bridge structure inspections are on the increase.

In the past it was sufficient to realize only visual examination of defects. Now the prognosis of a future behaviour of the structure and determination of a residual life-time are required. The new bridges are more frequently equipped with monitoring facilities including data acquisition systems. As a part of the bridge inspection is commonly used carbonation depth survey, content of chloride and extent of reinforcement corrosion. The most frequent methods for determination of corrosion extend are ultrasound, sonar, infra-red camera, radar, electric potential measurement, radiography, vibration analysis, etc.

Next potential methods, which can be used in this area, are modal analysis and acoustic emission method. The application of these methods on bridges is at the beginning.

The Czech Ministry of transport supports research and development (R&D) projects dealing with these two promising methods.

The article is dealing with R&D project, which is focused on usage of acoustic emission method as a tool for non-destructive testing of bridge structures, with a view to an assessment of reinforcement corrosion.

The measurement realized on laboratory samples and bridge structure will be mentioned, including information about equipment and measurement technique.

1. INTRODUCTION

Concrete and prestressed structures condition is affected by varied factors. One of the most negative factors is reinforcement corrosion. At present time, CDV - Transport Research Centre and Physical Department of Faculty of Civil Engineering at Brno University of Technology are dealing with development of a diagnostic method based on the principle of acoustic emission. This method should be used for structural defects monitoring of concrete and prestressed structures, especially bridges, where defects were caused by reinforcement corrosion.

2. ACOUSTIC EMISSION

Mechanical energy appears whenever any object or structure is stressed. The energy is emitted in the form of elastic waves. This phenomenon is generally called Acoustic Emission (AE). Normally operated bridges are affected by dynamic actions caused by transport, wind etc. That gives rise to the acoustic emission. The research is trying to find the characteristic frequencies ("unhealthy sound") that are generated by corroded reinforcement of bridges while they are stressed.

3. STATE OF THE ART

The research focused on usage of AE method in the field of testing of reinforcement corrosion and prestressed structures is currently in progress at some research centres all around the world [2]. It includes mainly testing of laboratory samples or testing of reduced girders, which are placed on the laboratory premises.

Canadian company Pure technologies has patented monitoring technology called Sound Print, which is able detect and localize wire breaks of prestressed cables with the accuracy of 0,5 m. The system uses an array of sensors, which monitor a part or whole supporting structure of bridges and similar structures. These sensors are connected with 32 channels acquisition unites. Signals are treated on-line and presented at Web pages. The provider of this technology in Europe is the company Advitam [3].

At Polish Kielce University they are dealing with a development of diagnostic method for testing of bridges based on AE principle. Loading of structures is realized by the help of heavy lories, statically or dynamically. AE signals are analyzed in time domain. They execute a zonal localization of areas of defects. To determine the condition of girders parts is used AE parameters analysis. On the base of these parameters they count other indicators called historic index and severity, whose combination indicates the degree of defects of individual parts of a structure, mainly girders [4].

The other possibility is to measure acoustic response during using of the bridge. At University of Edinburgh they realized in-situ long term monitoring of bridge condition with the view of detecting crack growth and determining position of the crack tips. They worked with time domain of the signal. The compared parameters were number of detected evens (their hits) and their energy [5].

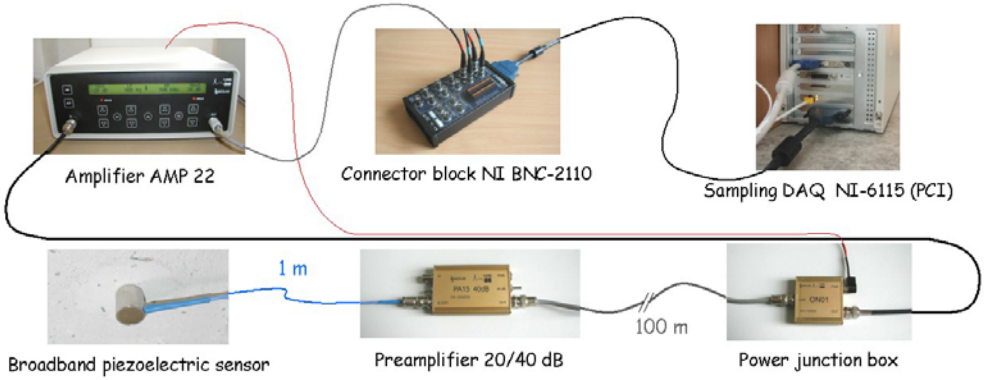

Fig. 1: Connection diagram for one channel of the measuring system

4. MEASURING EQUIPMENT

For the measurement purpose we have used four channel acquisition system containing of a efficient personal computer, 12-bit sampling card NI 6115, amplifiers AMP 22 and 31, preamplifiers PA 15 and broadband piesoelectrical sensors (up to 1 MHz). Recorded signals were processed by the composed CDV software and the results were presented by the help of NI Diadem software. Connection diagram for one channel of the measuring system is shown in figure 1. The basic parameters, which are count from the time domain of the signal are presented in figure 2. The other parameters are energy, root mean square, average signal level, average frequency, etc.

5. OWN RESEARCH

The aim of our research is to propose methodology for preparation and realization of measurement of reinforcement defects of concrete and prestressed structures, with a use of AE method. One-time measurements will be carried out with a help of mobile equipment. The proposed measurement system is supposed to allow repeated measurements of more structures because it is not fixed to one construction for a longer period of time. The results and conclusions will form a base for the proposal of monitoring, repair and maintenance system of concrete road bridges.

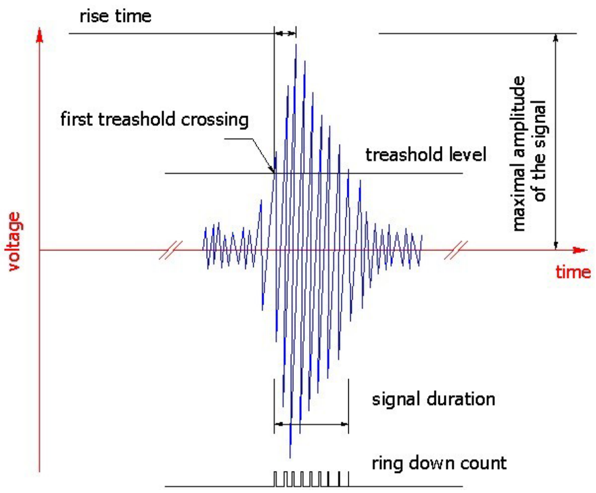

Fig. 2: Basic AE parameters counted from a time domain of the signal

6. LABORATORY MEASUREMENTS

Laboratory measurements were aimed to study an acoustic signal with the reinforcement corrosion correlation. The response signal to actuating pulse and AE signals recorded during the time when the samples were exposed to bending tension were analyzed.

In the first phase, the correlation of signal frequency spectra (response to actuating pulse on a reinforcement surface) for both corroded and partly corroded rebar was studied. There was an apparent shift of marked frequency components into lower frequency range in the case of partly corroded reinforcement.

In the second phase, the samples exposed to accelerated corrosion were repeatedly monitored. The response of these samples on the actuated pulse was processed again. The response was recorded simultaneously on the reinforcement and on the concrete surface of the sample. With increasing corrosion, in the frequency spectrum marked frequency components shift into lower frequency areas, see figure 3.

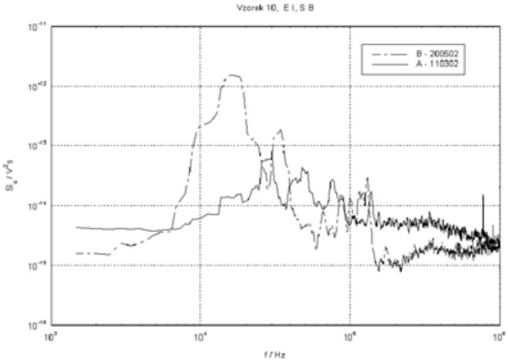

Fig. 3: Frequency spectra comparison

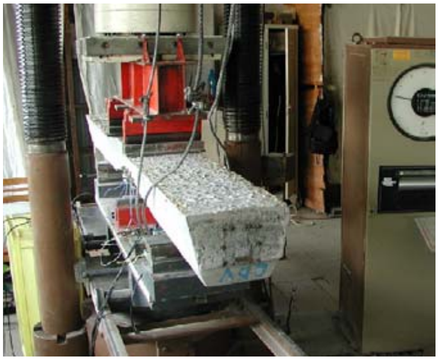

Fig. 4: Bending tension test

In the third phase, two sets of reinforced beams were monitored. One set aged in laboratory conditions while the other in an aggressive corrosive environment. Tested beams were repeatedly exposed to bending tensions according to European Standard EN 12390-5, see figure 4. Frequency spectra of the acoustic emission and the pulse frequency rate were processed.

The laboratory measurements proved correlation of changes in frequency spectra with structural changes caused by reinforcement corrosion. Satisfactory correlation was reached especially in the case of monitoring the proceeding corrosion by repeated sample measurements.

6. MEASUREMENTS ON THE BRIDGE

After verification of the possibility to detect corrosion of in-build reinforcement in laboratory conditions we started measurements in situ.

During the reconstruction of 20-year old bridge: 7-012 Brandysek, I73 bridge beams (prestressed I-beams) were tested by AE method. The bridge consisted of three spans with 9 beams, 30 m long each. Consecutively, when the beams were demolished, a condition of prestressed cables and construction reinforcement were checked.

6.1 Process of measurement

The bridge profile is shown in figure 5. Eighteen I 73 beams were measured with the AE method. Sensors were fixed at the ceiling of each beam in the middle of its length. AE signals were generated by the lorry travelling over a 15 cm high wooden chock. The chock was placed on the road surface exactly above the point of measurement. The measuring system was powered by a generator. The use of the platform is shown in figure 6, AE signals generated by the lorry travelling over the chock is shown in figure 7.

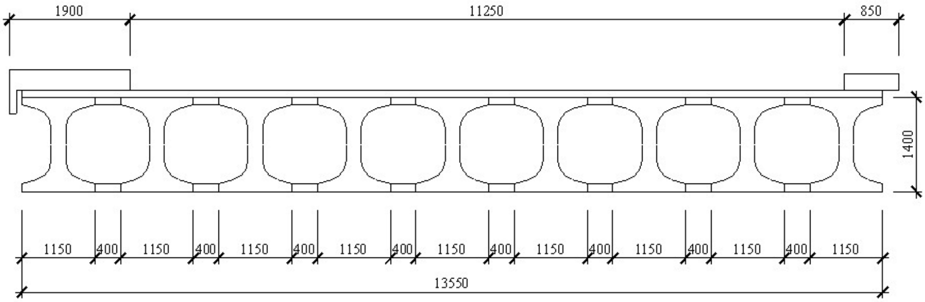

Fig. 5: Brandysek bridge profile

During the whole time the lorry travelling over the chock AE signal was continually recorded (approx. 15 seconds). Sampling frequency was selected at 1 MHz. Each beam was measured repeatedly, three times.

Fig. 6: Sensors placement process

Fig. 7: Lorry travelling over the chock

6.2 Beams demolition

Even before our measurement a decision was taken that the existing I73 beams would be removed and replaced with steel beams. The beams demolition was carried out on the spot with a help of a pneumatic hammer and hydraulic cutting shears. With the help of this equipment the concrete was broken, construction and prestressed reinforcement separated and taken away to be recycled. A visual examination was carried out both at the point of measurement and also along the whole length of the beam including the anchors and Sandrik steel pipes. The examination was aimed to all prestressed reinforcement consisting of 20-wire cables of 4.5 mm in diameter.

There was no significant effect of corrosion on the reinforcement section. For the evaluating purposes, the corrosive attack extent was divided into 4 groups (k1 up to k4).

Fig. 8: Four groups of corrosion

The reinforcement basally untouched by corrosion was put into the first group k1. The reinforcement close to the anchors, which was weakened by corrosion up to 0.5 mm and with initial signs of penetrating corrosion was placed into the fourth group k4, see figure 8.

6.3 The beams evaluation

FFT (Fast Fourier Transformation) was used to transform the acoustic signal from the time domain to the frequency spectrum.

The typical recorded signal is shown in figure 9. The arrows indicate events occurred when the lorry wheels hit the road surface. The arrow 1 corresponds to the front axle, the arrows 2 and 3 correspond to the wheels of the rear axle.

The time domain corresponding to the second and third wheel hitting were selected to analyze signals of individual beams. In figure 10, there is shown a time domain corresponding to hitting the second wheel of the rear axle to the road surface. Figure 11 represents its frequency spectrum.

There were found no significant differences when all frequency spectra were compared, in a range between 1 kHz and 500 kHz. In the scope of performed measurements, no frequency was found that would indicate some significant construction defect caused by the reinforcement corrosion. That corresponds to the visual examination of the reinforcement at the points of measurement. It proved that both the prestressed and the construction reinforcement were in suitable condition concerning to the corrosion.

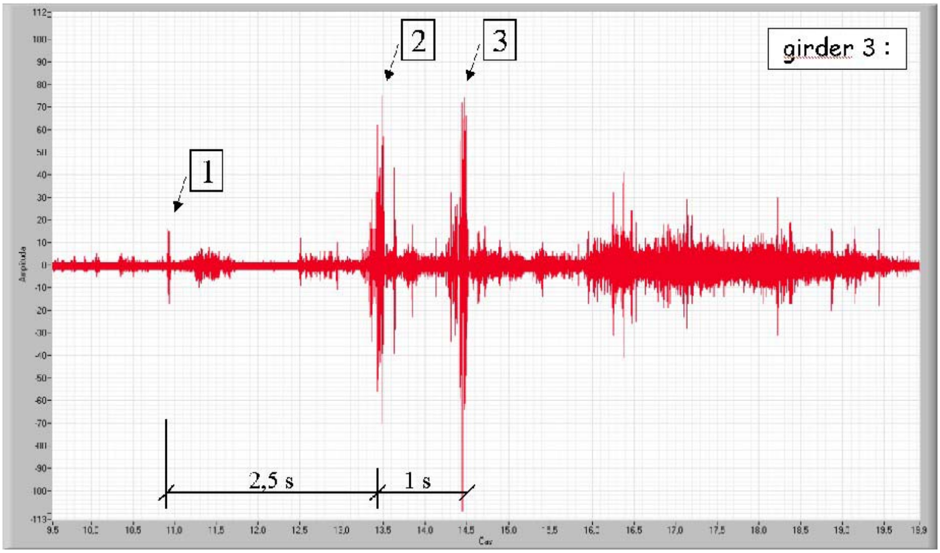

Fig. 9: Whole signal in time domain

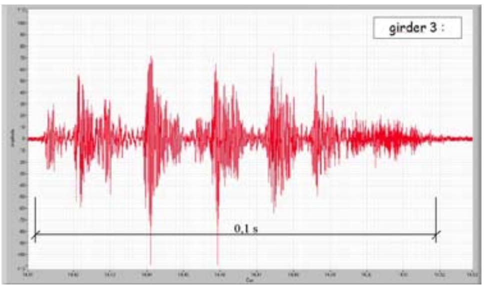

Fig. 10: Wheel 2 - time domain

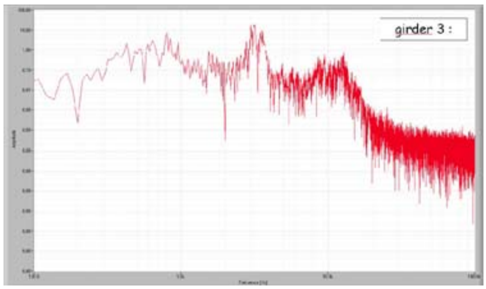

Fig. 11: Wheel 2 – frequency spectrum

7. CONCLUSION

The significant advantage of the AE method is the fact that it indicates the unstable and dangerous defects, which are active at the given structure stress. In comparison with the other non-destructive diagnostic methods AE method can give global information about the examined object condition. According to the AE measurement results, it is possible to aim a diagnostic checking to those points where the emission sources were detected preferentially. Thus the AE method becomes a useful supplement to the common diagnostic methods.

Acknowledgements

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Technology Research in Transport Infrastructure

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INTRODUCTION

Due to the continuing decrease in the amount of material and energy sources during the recent years, as well as the increase of their prices, there has been more emphasis on the reliability, lifespan and safety of transport infrastructure constructions, while minimizing their negative effects on the living environment. Therefore the Transport Research Centre (Centrum dopravního výzkumu - CDV), specifically the Infrastructure Department, has been trying to help this trend by a research in the field of geotechnics, technology of concrete and non-destructive structures testing.

1. GEOTECHNICS

In the field of geotechnics the research activity concentrates on projects linked to the relationship between geotechnical quantities and geosynthetics influence on increase of soft soil bearing capacity. In order to make the geotechnical research more effective, a Geotechnical Laboratory Testing Field (GLTF) was built in the year 2001.

1.1. Geotechnical Laboratory Testing Field (GLTF)

The GLTF is a laboratory tool, which allows the measurement, in a laboratory, of some of the geotechnical quantities usually measured in the field (such as a plate test, dynamic loading test, penetration test, etc.) on various soils and soil layers for different compaction rate and water regimes. Unquestionable advantage of GLTF is the possibility to carry out all kinds of geotechnical tests on constructions of real scale while still in the laboratory-protected conditions.

The GLTF, Figure 1 left, consists of concrete pit split by removable dividers into separated measuring (testing) spaces and a watering/dewatering drain channel separated by removable dividers too. There is a drain layer placed on the bottom of each measuring space closed off by a grate with a geotextile drainage filter. Both the concrete pit and the drain channel are interconnected at their bottoms. A moveable frame can be slid in a longitudinal direction along guide rails fastened to the top of the pit. The moveable frame serves for mounting or supporting of measuring equipment (plate test, CBR in situ test equipment etc.) and can be locked in both horizontal and vertical senses during testing.

The GLTF has been recently equipped with loaders for accelerated testing of soil and unbound layers, see Figure 1 right. The GLTF is becoming accelerated loading testing equipment. Currently the geosynthetics behaviour under cyclic loading (simulation of traffic loading) is tested. There is also a plan for a research project focused on the accelerated testing of reinforced joins. The project should be aimed on tolerance findings of reinforcing dowels angular displace and durability of additionally placed dowels.

The GLTF description has been published [1,2] and is protected by the utility design [3].

Fig. 1 – Laboratory Geotechnical Testing Field (GLTF) left – general view with static plate test, right – equipment for cyclic loading

1.2 Correlation of California Bearing Ratio CBR and deformation modulus

Deformation modulus, especially its value detected by static loading test in the second loading cycle E_{v2}, is one of the most important parameters checked at the final subgrade, just after the compaction rate and moisture content of the soil. Deformation modulus is used for verifying deformation characteristics of final subgrades. In a number of European countries, e.g. in Germany, Austria, Czech republic, Slovakia, etc. there is a minimum value of this modulus specified by standards, which has to be reached.

Table 1 – Deformation Modulus of the second cycle $E_{def,2}$ for various soil types

Serial No.	Soil type	Symbol	Fine particle content f [%]	California Bearing Ratio *CBR* [%]		Modulus E_{v2} [MPa]
				Optimum Moisture	95% saturation	
1	gravel silt	F1 MG	35 – 65	8 – 18	5 – 10	20 - 40
2	gravel clay	F2 CG	35 – 65	5 – 10	3 – 7	15 - 30
3	sandy silt I	F3 MS_1	35 – 50	5 – 25	4 – 15	15 - 45
4	sandy silt II	F3 MS_2	50 – 65	3 – 15	2 – 5	5 - 40
5	sandy clay I	F4 CS_1	35 – 50	5 – 30	5 – 20	15 - 50
6	sandy clay II	F4 CS_2	50 – 65	2 – 20	0 – 4	0 - 40
7	low plasticity silt	F5 ML	> 65	2 – 20	2 – 7	0 - 40
8	medium plasticity silt	F5 MI	> 65	2 – 15	1 – 6	0 - 40
9	low plasticity (lean) clay	F6 CL	> 65	3 – 20	1 – 8	5 - 40
10	medium plasticity clay	F6 CI	> 65	2 – 20	0 – 6	0 – 40
11	high plasticity silt	F7 MH	> 65	3 – 7	0 – 4	5 – 25
12	very high plasticity silt	F7 MV	> 65	2 – 6	0 – 3	0 – 20
13	extremely high plasticity silt	F7 ME	> 65	2 – 5	0 – 2	0 – 20
14	high plasticity clay	F8 CH	> 65	2 – 7	0 – 3	0 – 25
15	very high plasticity clay	F8 CV	> 65	1 – 7	0 – 3	0 – 25
16	extremely high plasticity clay	F8 CE	> 65	1 – 6	0 – 3	0 – 20
17	well-graded sand	S1 SW	< 5	-	-	-
18	poorly-graded sand	S2 SP	< 5	-	-	-
19	sand with fine soil additive	S3 S-F	5 – 15	8 – 70	6 – 25	20 – 65
20	silty sand	S4 SM	15 – 35	6 – 50	4 – 15	15 – 55
21	clayey sand	S5 SC	15 – 35	4 – 30	2 – 12	10 - 50
22	well-graded gravel	G1 GW	< 5	-	-	-
23	poorly-graded gravel	G2 GP	< 5	-	-	-
24	gravel with fine-soil additive	G3 G-F	5 – 15	20 – 90	6 – 60	35 - 65
25	silty gravel	G4 GM	15 – 35	10 – 60	4 – 40	25 - 60
26	clayey gravel	G5 GC	15 – 35	5 – 30	3 – 20	15 - 50

Predictability of the compliance with a specific minimum value of the deformation modulus is a crucial condition for the economy proposal for the earth works, as the real value of this modulus can be detected only after the earth works have been finished. The final subgrade, or the capping layer, then either meets or doesn’t meet the given criteria. If the deformation modulus detected at the given deformation and moisture ratio is lower then required, the subgrade has to be usually remoulded, adjusted and then compacted again. It is obvious that this causes undesired extra expenses.

Table 2 Measurement on Subgrade with Bearing Capacity of 5 MPa

Testing space	Geosynthetics	Layer thickness	Modulus E_0 [MPa]	Bearing capacity increase	Modulus E_{v2} [MPa]	Bearing capacity increase
I	welded geogrid	15 cm	11.38	1.22	11.38	1.13
		30 cm	23.65	1.07	22.06	1.13
II	non-woven geotextile	15 cm	9.08	no benefit	9.94	no benefit
		30 cm	22.74	no benefit	17.43	no benefit
III	unreinforced (referential)	15 cm	11.13	1.00	10.11	1.00
		30 cm	22.21	1.00	18.18	1.00

Table 3 Measurement on Subgrade with Bearing Capacity of 7 MPa

Testing space	Geosynthetics	Layer thickness	Modulus E_0 [MPa]	Bearing capacity increase	Modulus E_{v2} [MPa]	Bearing capacity increase
I	unreinforced (reference)	20 cm	17.90	1.00	unexecuted	N/A
		40 cm	69.16	1.00	53.66	1.00
II	woven geotextile	20 cm	23.23	1.30	unexecuted	N/A
		40 cm	63.76	no benefit	56.51	1.05
III	rigid Geogrid A	20 cm	24.89	1.39	unexecuted	N/A
		40 cm	62.08	no benefit	55.41	no benefit

From the problem described above it can be seen that it is useful to know the expected deformation modulus of the subgrade already at the time of the proposal, given the specific soil type; or alternatively to know what sort of soil adjustment should be suggested so that the expected value of the deformation modulus is sufficient. As can be observed from the professional studies, one of the means for this prediction can be the correlation of the deformation modulus and the California Bearing Ratio (CBR). Unfortunately, in the literature available, no relation between the deformation modulus in the second load cycle and CBR specified at the same or optimum moisture content has been found.

Assumptions, methodology and evaluation from the research described here have been continuously publicized, e.g. [4, 5] (they are also available from the author of the English version). So it would be only superfluous to give here their detail description here, and moreover it would unproportionately excess the capacity of this article. However it should be state here that the tests have been carried out on real construction sites and in GLTF, on various soil types, on various compaction and moisture. The California Bearing Ratio *CBR* has been detected at intact samples, which were collected within a minimum distance from the compaction modulus testing place. Thus both of the correlated geotechnical quantities (*CBR* and E_{v2}) have been detected under the very same conditions (moisture, compaction ratio, etc.).

Table 4 Measurement on Subgrade with Bearing Capacity of 15 MPa – 1st Series

Testing space	Geosynthetics	Layer thickness	Modulus E_0 [MPa]	Bearing capacity increase	Modulus E_{v2} [MPa]	Bearing capacity increase
I	Woven geotextile	20 cm	32.35	no benefit	25.17	no benefit
		30 cm	43.75	no benefit	33.28	no benefit
II	rigid geogrid A	20 cm	31.90	no benefit	29.79	no benefit
		30 cm	47.06	no benefit	34.06	no benefit
III	unreinforced (reference)	20 cm	31.74	1.00	28.43	1.00
		30 cm	47.35	1.00	34.20	1.00

Table 5 Measurement on Subgrade with Bearing Capacity of 15 MPa – 2nd Series

Testing space	Geosynthetics	Layer thickness	Modulus E_0 [MPa]	Bearing capacity increase	Modulus E_{v2} [MPa]	Bearing capacity increase
I	rigid geogrid B	20 cm	32.98	no benefit	25.94	no benefit
		30 cm	46.48	no benefit	35.19	no benefit
II	welded geogrid	20 cm	32.80	no benefit	27.87	no benefit
		30 cm	47.38	no benefit	35.54	no benefit
III	unreinforced (reference)	20 cm	38.33	1.00	29.26	1.00
		30 cm	52.48	1.00	37.74	1.00

One of the solution output is table 1. This is in a certain modification also included in the draft revision proposal of the Technical Conditions of the Transport Ministry TP 77 – Road design whose elaborator is the Faculty of Civil Engineering, Brno University of Technology. Table 1 has been done by compiling the results of the CDV research (column E_{v2}) and the enclosures from the Czech Standard CSN 72 1002.

1.3 Study of the geosynthetics effect on the increase of the bearing capacity of subgrade

Geosynthetics (geotextiles, geogrids, geonets, geocells and geocomposits) are suggested as substances for improving the characteristics of materials of constructions used in transport civil engineering. Every geosynthetic should fulfil at least one or more of the following functions: filtration, separation, draining, protection, anti-erosive and reinforcing function. The last mentioned reinforcing function used for example in case of embankment construction when steeper slope of embankment can be designed is determined relatively well. While in the literature, especially in the company materials, one can often find information that by using geosynthetics one can be increased even the bearing capacity of soft soil, the problem has not been solved satisfactorily yet.

The research carried out at CDV concentrated especially on the study of the geosynthetics' effect on the increase of the bearing capacity soils. A number of validation tests have been carried out in GLTF recently. The specialists/professional public have been informed about the continuous results through journal and conference papers, for example see [6, 7]. The quoted publications discuss in detail the results of the surveys carried out. For this reason, it is described here the research resume only.

Presumptions and parameters

Measurements of the bearing capacity were done by modulus of deformation obtained from the second loading cycle of the static plate test, see Figure 1 left, according to two methods. The first method is widely used in Europe for highway subgrade evaluation and it is described some European standards, e.g. in DIN 18134 (German Standard) of CSN 72 1006, Appendix A (Czech Standard), and the resulting modulus is called E_{v2}. The second method used for railway subgrade evaluation is described in Czech Railway Standard S4, and the resulting modulus is called E_0. Both methods vary mainly in the ways of loading, and there is also a small dissimilarity in the modulus of deformation calculation. However, for our purpose, it is possible to say that both methods are able to express the impact of geosynthetics on the bearing capacity in a similar way.

The GLTF was divided into 3 testing spaces for measurement and experiments to be performed according to the following conditions. A 70cm thick bed of loess, compacted layer by layer, simulating a soft-soil subgrade was placed on the drainage layer of all three testing spaces of the GLTF. The compaction of the soft soil was about 95 % Proctor Standard (PS) and its bearing capacity was set by moisture content on 5 MPa, 7 MPa and 15 MPa respectively.

Selected geosynthetics were laid down in to two GLTF testing spaces and one testing space was kept without geosynthetics for comparison. After that, a crusher-run material was spread as a sub-base layer. It was placed in 15cm or 20cm thick layers, and compacted to at least I_D = 0.85, as measured. After measuring the modulus on the top of sub-base layer, an additional 10cm, 15cm, or 20cm thick sub-base layer was spread and the measurement was repeated.

Measurement and results

As indicated above, measurements were carried out in the GLTF, which had been divided into the three same size (3 m x 3 m) testing spaces. Two geosynthetics were measured in one step – one by one in each of two testing spaces and one testing space was kept without geosynthetics a for comparison. Measurement of moduli was carried out three times in each testing space according to both highway and railway standards mentioned above. The Table 2 to 6 display average values of both moduli measured on 15 cm or 20 cm level and on 30 or 40 cm level of sub-base.

Table 6 Measurement on Subgrade with Bearing Capacity of 15 MPa – 3rd Series

Testing space	Geosynthetics	Layer thickness	Modulus E_0 [MPa]	Bearing capacity increase	Modulus E_{v2} [MPa]	Bearing capacity increase
I	flexible geogrid A	20 cm	31,16	no benefit	27,18	no benefit
		30 cm	47,92	no benefit	34,82	no benefit
II	flexible geogrid B	20 cm	29,99	no benefit	29,79	no benefit
		30 cm	42,26	no benefit	37,06	no benefit
III	unreinforced (reference)	20 cm	33,36	1.00	28,43	1.00
		30 cm	49,95	1.00	39,89	1.00

Various kinds of woven and non-woven geotextiles, and welded, flexible and rigid geogrids were used for the experiment. All of them are products of well-known producers and are certified for highway and railway usage.

Discussion of results

Table 2 displays measurement results for welded geogrid and non-woven geotextile on a very weak subgrade of 5 MPa of bearing capacity. Measured data shows that there is no benefit from non-woven geotextile on bearing capacity increase. However, welded geogrid demonstrated 22 % and 13 % (railway and highway modulus measurement methodology) bearing capacity increase on a 15cm thick sub-base layer in comparison with unreinforced structure, and 7 % and 13 % bearing capacity increase on a 30cm thick sub-base layer.

Table 3 displays measurement results of woven geotextile and rigid geogrid A on a very weak subgrade of 7 MPa of bearing capacity. Measured data shows that woven geotextile yields bearing capacity increases of up to 30 %, and rigid geogrid A up to 39 % on a 20cm thick sub-base layer; however there is no significant bearing capacity increase on a 40cm thick sub-base layer.

Tables 4, 5 and 6 display measurement results obtained for woven geotextile, rigid geogrid A, rigid geogrid B, welded geogrid, flexible geogrid A and flexible geogrid B placed on weak subgrade of 15 MPa of bearing capacity. All results show that there is no significant bearing capacity increase on a 20cm and 30cm thick sub-base layer either.

Outcome

The performed experiments imply that the influence of selected geosynthetics on the bearing capacity increase of weak subgrade is very limited. Measurement shows geosynthetics are able to increase the bearing capacity of very weak soil (a subgrade with bearing capacity expressed by deformation modulus of 5 MPa or 7 MPa), namely in relation to a relatively thin sub-base layer – up to 20 cm. As such a weak subgrade is not useful for highway or railway foundation, it would be useful only for the temporary subgrade improvement of roads on such a weak subgrade. Improvement of weak soil (bearing capacity 15 MPa) by geosyntetics was not shown, in contradiction to the claims in the trade publications from the geosynthetic producers.

2. SELF-COMPACTING CONCRETE (SCC)

A very important operation carried out in case of monolithic concrete constructions and during the production of prefabricated elements is the compaction of the concrete mixture. Compaction is there to ensure both the required density of the concrete, its homogeneity, and also infilling of all the specified room with the concrete mixture, so that the synergism of the concrete with the reinforcement is ensured too. A number of compaction methods and compacting devices can be used for this purpose achievement. Especially nowadays when more and more subtle constructions with high reinforcement degree are being designed, the execution of the compaction process is becoming more demanding, as it is difficult to ensure optimum compaction of all parts of the future construction or of the prefabricated element. This can then cause cavities, gravel nests or other non-uniformity signs or anisotropy, which may degrade the visual effect, allow reinforcement corrosion or endanger statistic or possibly even dynamic characteristics of the whole construction. A modern solution to the problem described is using such a concrete mixture that will fill in the entire volume of the construction, thanks to its own gravity effect, and at the same time completely coat up the reinforcement - all this without any need of a compaction operation. The material of such characteristics is called the Self-Compaction Concrete (SCC). But together with the SCC technology being introduced to the civil engineering practice a number of other problems that didn’t used to be so important in the past when using traditional concrete technologies become now topical. Mixtures for self-compaction concrete must show high level of mobility at appropriate viscosity (a mixture is supposed to fill in the entire designated space spontaneously), there can be no segregation of the coarse components in the mixtures caused by the effects of the components mobility or by blocking the reinforcement, etc. However, compliance with these requirements often affects the fundamental concrete parameter, which is its strength.

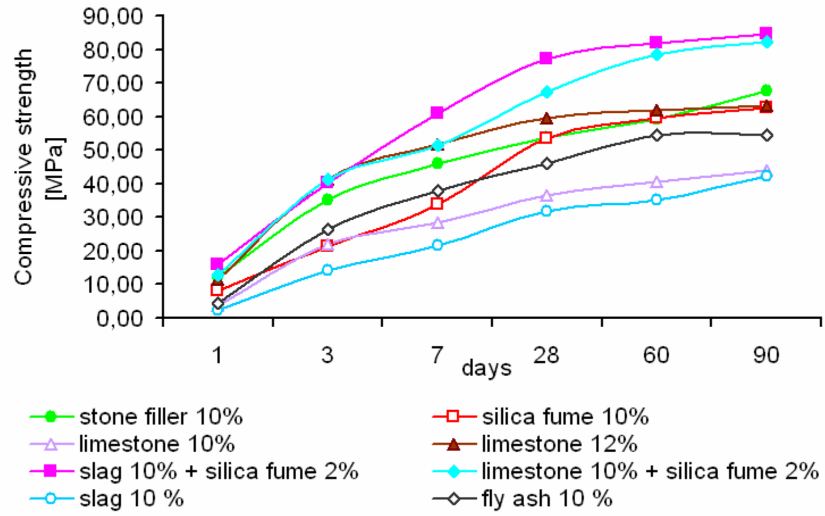

Fig. 2 – Development of the compression resistance increase of various SCC in relation to time

One of the means for reaching the required characteristics of the mixture is making use of the new generation surface-active substances, i.e. super-plastificators adjusting the mixture mobility, and also by the means of higher volume of fine particles proportion in the filler, ensured by the addition of micro-fillers. Industrial by-products can be used as the micro-fillers (e.g. silica fume, fly ash, finely grained slag or limestone, stone fillers, etc.). The SCC technology is therefore important also from the view of utilising waste materials and thus contributing to the environment protection.

The SCC technology development has been known since the last decade of the 20th century, so it is a rather new technology. However, CDV infrastructure laboratories have been carrying our surveys concentrated on monitoring the effects of concrete mixture composition on the characteristics of self-compaction concrete already since the year 2000.

Our tests have proved significant behavioural changes of concrete mixtures in relation to their type and volume of the used micro-filler. Similarly, the results of tests on the matured concrete prove a significant dependence of their mechanically-physical parameters on the formulation of the used concrete mixture, see for example the graph in fig. 2. The conclusions of some experimental tests have already been published [9, 10, 11]. Further tests are currently taking place and their results will be gradually announced to the specialists.

3. NON-DESTRUCTIVE TESTS OF BRIDGE CONSTRUCTIONS

The analysis of the central bridge registration on the roads of the CR shows that from the total number of 15650 of road bridges only 5823 conform or conditionally conform (i.e. almost don’t conform), which is more than a third (37%). From the mentioned number further 1 360 bridges are dilapidated (in a state of disrepair). 19 of these bridges have their span bigger than 100 m, 97 of bridges have span 30 - 100 m, etc. Almost the same can be said about the conditions of bridges in other European countries.

Such poor bridge conditions are caused partly by the lack of financial means for their maintenance, but also by the absence of adequate, relatively quick and cheap monitoring method which would enable us to detect their defects in early stages, allowing us simple a and financially not very demanding maintenance. One of such possible solutions could be a method based on the principle of acoustic emission (AE). Diagnostic methods using AE belong to the group of non-destructive passive methods and use gradual wave pulses. The signals of acoustic emissions accompany the dynamical processes in the material and then come through as gradual elastic wave motion. The source of such wave packages are sudden energy releases in the material. This process is followed by deformation, fracture or phase changes in the material.

CDV is, together with the Physics Institute of Faculty of Civil Engineering, Brno University of Technology, the solver of the grant of the Czech Ministry of Transport “Methodology of the reinforcement corrosion process determination of reinforced and prestressed structures”. The aim of this project is to produce technical guide that will codify the acoustic emission (AE) method as a method for routine use in the bridge management system.

The survey began by a detail study of survey results of other prominent research centres. It was realised that the task given by the Ministry was with its approach to the point of issue entirely new and that now other research centre had ever specialised in such a project. So from this point of view, the survey is quite unique, on the border of basic research and applied survey. And even though the AE principle has been know for several years, it has never been quite verified whether the corrosion process in the reinforcement can come through in the frequency spectrum of the construction.

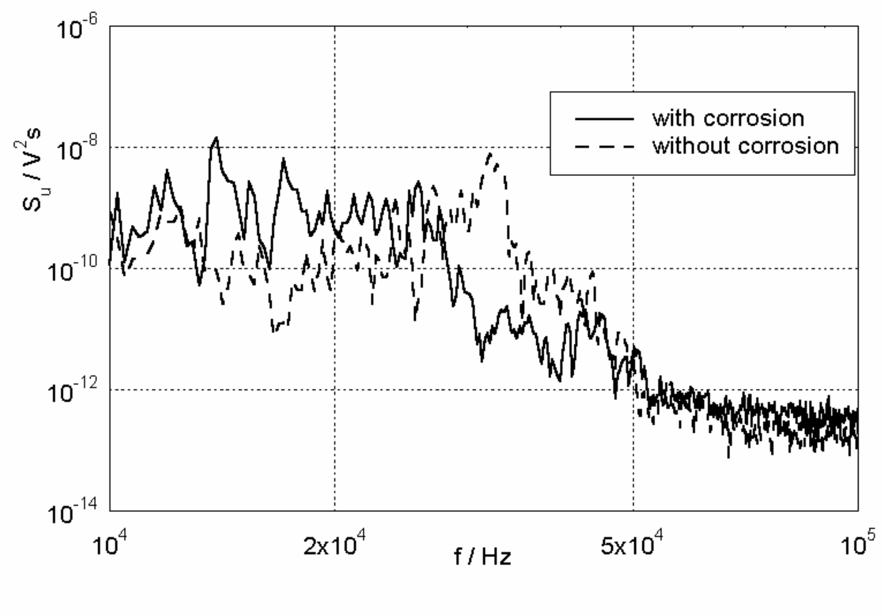

Fig. 3 – Comparison of spectra of supports with corroded and not corroded reinforcement

Another part of the survey was dedicated to finding a device that would be suitable for reading the AE signals both in laboratories and in real conditions. Firstly, a two-channel device was constructed which was later extended into a four-channel one. So now we can measure a signal read by four indicators placed at different parts of the construction at the same time. First measuring was carried out on iron-concrete samples of the size of 10x10x40 cm with built-in corroded and not corroded steel reinforcement of 8 and 10 mm in diameter.

Fig. 4 – Testing on bridges

The AE impulses have been induced by hits with special hammer onto the surface of the concrete samples, later induced with dynamic loading in a loading presser, which complies better with the real conditions. From the analysis of signals gained when using the Fourier transformation we can see the difference of frequency characteristics between samples with the corroded reinforcement and samples with non-corroded reinforcement, see for example fig. 3.

The laboratory tests were followed by tests on real bridge constructions. The tests have so far been carried out on 5 bridges at different places of the Czech Republic. The AE signals have been induced by driving a dumping car Tatra over a wooden 15 cm high sill. The indicators have been placed on the bridge ceiling, see fig. 4. The results of the research are being continuously published, e.g. [12, 13].

4. CONCLUSION

The activity description of CDV in the field of the transport infrastructure technology mentioned in this article is obviously not complete, due to the limited extent of this article. So let me allow to give at least a simple list of other CDV activities. CDV currently or recently is or was working on other research projects: together with companies PONTEX, Motorway structures Prague (Dálniční stavby Praha) and SMP Construction, on a project of the Ministry of Transport (MD) "Cement concrete pavements on bridges", see for example [14], and also on various projects of COST 343 – Reduction in Road Closures by Improved Pavement Maintenance Procedures, COST 344 – Improvement to Snow and Ice Control on European Roads and Bridges, COST 347 – Improvement in Pavement Research with Accelerated Load Testing, COST 351 Water Movement in Road Pavements and Embankments, COST 353 – Winter Service Strategies for European Road Safety, COST 354 – Performance Indicators for Road Pavements, TREE – Transport Research Equipment in Europe, etc.

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Determination of the carbon / hydrogen ratio in bitumen using prompt neutron gamma activation analysis

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Abstract

*The paper presents a potential application of PGNAA method that allows the fast determination of colloidal index of bitumen compounds by a very fast analysis of hydrogen and carbon content. The H/C ratio thus determined is then correlated with colloidal index I_C. The regression line is given by the empirical relation C/H = 5.9+ 34.1*I_C with a correlation factor R=0.99 and a standard deviation SD = 0.71. The perspectives of on-line applications are discussed.*

1. INTRODUCTION

Bitumen is very important industrial material whose quality is connected with its chemical composition, which is controlled mainly by the crude oil processing technology. Though it comprises a great number of chemical compounds the following model appear to be in general accepted: The high molecular weight asphaltenes which tend to form sterical colloids dispersed in a lower molecular weight oily medium (maltenes). This "see" of maltenes in its turn is constituted from saturates, aromatics, and resins. The proportions of these molecular groups give the physical properties of various bitumen samples /1,2/. The colloidal index of bitumen is defined as a ratio of the total amount of asphaltenes and saturates to the amount of resins and aromatics. It describes the stability of colloidal structure.

Using a Prompt Gamma Neutron Activation Analysis (PGNAA) Set Up developed and installed at IFIN-HH; Bucharest we present an industrial application of this nuclear method devoted to a fast estimation of colloidal index of bitumen.

2. THE EXPERIMENTAL SET-UP

The experimental set-up consists of an Am-Be neutron source (10^7 n/s) placed in a cylindrical vessel and a semiconductor Ge-Li detector outside the vessel. The experimental arrangement is presented in fig. 1. The prompt gamma spectrum is recorded and analysed using an integrated computerised system. The energy calibration was carried out using the known prominent gamma lines in the different parts of the spectrum. The MCA soft provides the digital spectrum stabilization (DSS) by choosing a few known references peaks.

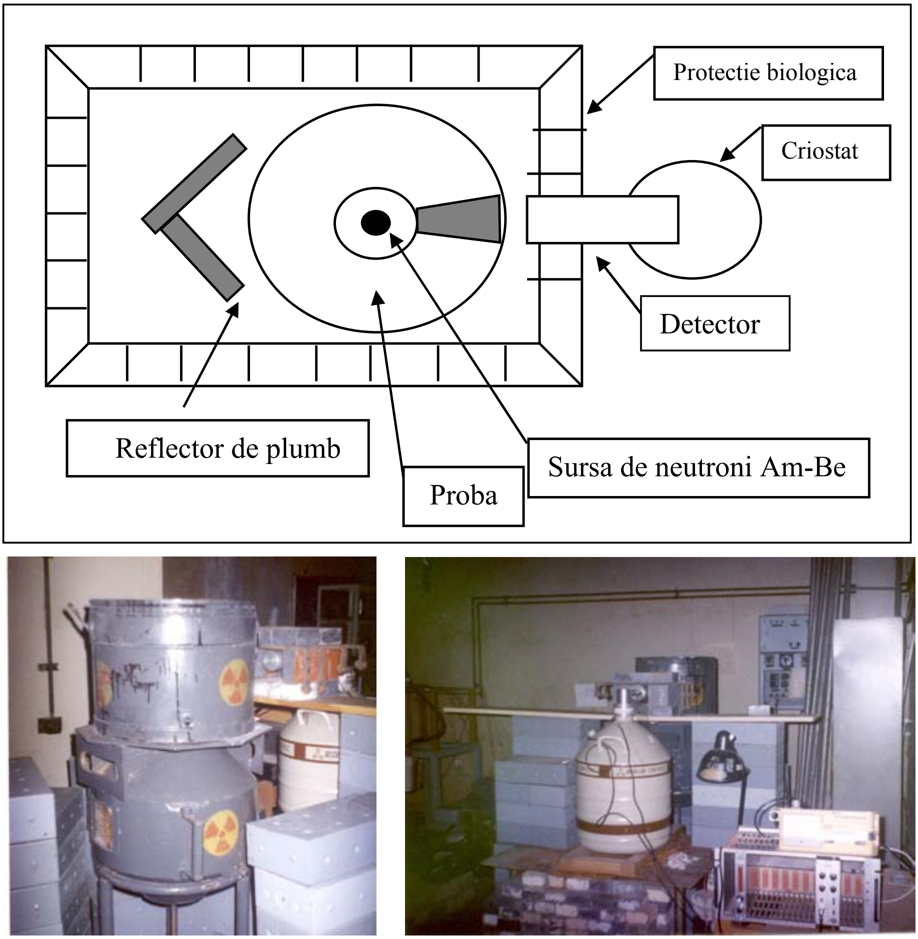

Fig. 1. PGNAA Schematic and view for experimental set up

3. PGNAA SPECTRA

The comments of the gamma lines obtained in such a geometry can be found in /3/.The PGNAA spectra were determined for three types of bitumen compounds. For exemplification in fig.2 the gamma spectrum for an ESSO sample is shown.

The measuring time was one hour and the dead time was about 5 %. The inelastic scattering carbon lines are Doppler broadened.

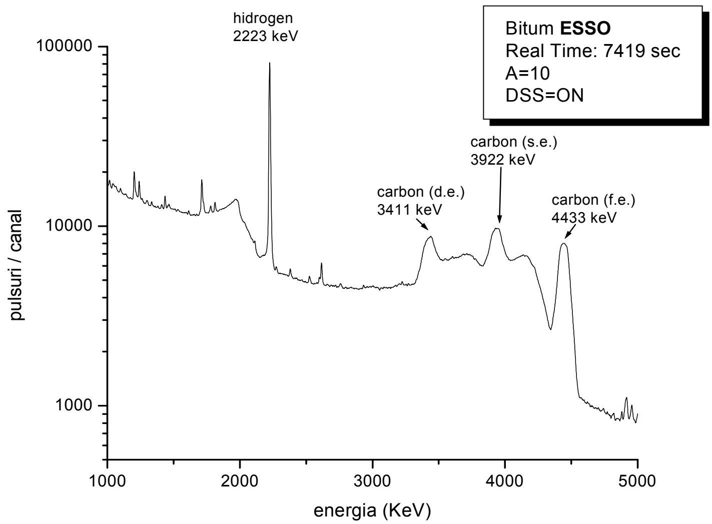

Fig.2. The PGNAA spectrum for ESSO bitumen sample

4. C/H RATIO

In order to determine the C/H ratio for bitumen by PGNAA method we used the thermal capture line of hydrogen at 2.22 Mev and the inelastic scattering (n, n’,γ) of carbon at 4.43 Mev. The concentration of hydrogen will be given by:

$$\nu_H = \frac{A_H . R_H}{\varepsilon(E_{H,\gamma}).I_{H,\gamma}.\sigma_{\gamma,H}} \tag{1}$$

where A_H stands for atomic mass, R_H-the peak area of hydrogen capture line, ε(E) is the detector efficiency at energy E of the line, I-the photon emission probability per captured neutron, while σ stands for thermal neutron capture cross section of hydrogen. In the case of carbon we deal with an averaged cross section on the fast neutron flux that is present in the sample:

$$\nu_C = \frac{A_C . R_C}{I_C . \varepsilon(E_{C,\gamma})} \tag{2}$$

where A_C is the mass of the carbon nucleus , R_C is the carbon line area, ,ε(E) is the detector efficiency at 4.43 Mev while I_C, the sensitivity factor is given by the following integral

$$I_C = \int_0^{E_{\max}} S'(E).\sigma(E).dE \tag{3}$$

It represents the averaged cross section on effective fast neutron flux S'(E) normalised to unity. The value of this integral for an Am-Be neutron source placed in moderator medium, composed especially from carbon and hydrogen (like hydrocarbons, coal, or polymers) was calculated by Wormald.

$$I_C = 0{,}11029$$

The aromatic factor C/H will be given by the ν_C/ν_H ratio:

$$\frac{C}{H} = \frac{A_C.R_C}{A_H.R_H}\frac{\varepsilon(E_H)}{\varepsilon(E_C)}\frac{I_C}{\sigma_{\gamma,H}} \tag{4}$$

Here the value of σ_H is 0,332 barn, A_C = 12, A_H= 1, while the efficiencies ratio $\varepsilon(E_H)$ / $\varepsilon(E_C)$ was obtained from the efficiency function of Ge-Li detector calculated by us by Monte Carlo simulations:

$$\frac{\varepsilon(E_H)}{\varepsilon(E_C)} = 1{,}23$$

The calibration of the experimental system was realized by using an graphite-alum mixture:

$$(NH_4)\ Al.(SO_4)_2\ 12.H_2O$$

This compound contains:

H	6.2%
O	70.6%
N	3%
S	14.0%
Al	6.2%

The measurements were carried out on two such mixtures:

50% graphite	50% alum	C/H ratio = 16,13
70% graphite	30% alum	C/H ratio =24.19

In order to have a good homogeneity the two materials were very well chewed and the geometry of the system was kept identical. On the other hand the bitumen has a composition dominated by carbon (81÷86%), hydrogen (9.5÷10.8%), and sulfur (1.3÷6.9%).The comparison of PGNAA data with real mixture composition gave the following results:

Table 1. The main data of the calibration mixtures for check

Sample Number	Graphite / Alum ratio	C/H ratio mixture	C/H ratio PGNAA	Deviation (%)
1	50(%) / 50(%)	16,13	15,54	3,6
2	60(%) / 40(%)	24,19	25,48	5,06

The aromatic factor C/H will be given by the ν_C/ν_H ratio:

$$\frac{C}{H} = \frac{A_C.R_C}{A_H.R_H}\frac{\varepsilon(E_H)}{\varepsilon(E_C)}\frac{I_C}{\sigma_{\gamma,H}} \tag{5}$$

In the last relation σ_H is 0.332 barn, $A_C = 12$, $A_H = 1$, while the ratio $\varepsilon(E_H) / \varepsilon(E_C)$ was evaluated from Monte Carlo simulations

$$\frac{\varepsilon(E_H)}{\varepsilon(E_C)} = 1,119 \text{ (for second escape line of carbon)}$$

Replacing these values in (5) we obtain a very simple empirical formula for the evaluation of the aromatic factor C/H.

$$\frac{C}{H} = 40.5 \times \frac{R_C}{R_H} \tag{6}$$

For a given experimental arrangement the relation (6) allows the fast determination of this parameter that characterises the bitumen sample. This can be correlated with other technical parameters like viscosity, thermal susceptibility, and hardness if the correlation functions are apriori known. In the present work we correlated this ratio with colloidal index I_C , a technical parameter that is connected with bitumen composition that in its turn gives the microscopic structure. /1,2/. It is defined as a ratio of the total amount of asphaltenes and saturates to the amount of resins and aromatics. It describes the stability of colloidal structure.

5. COLLOIDAL INDEX

Bitumen is composed generally by carbon (81÷86%), hydrogen (9.5÷10.8%), sulphur. (1.3÷6.9%). Small amounts of other elements are also present: oxygen, (1÷2 %), nitrogen (1÷2 %). Though the hydrogen content of bitumen is greater than in the case of our calibration mixture we used the same value of integral I_C. We can not appreciate the errors of this effect. The results are presented in table 2.

Table 2. The results of PGNAA analysis correlated with colloidal index.

Sample	Hydrogen peak area (counts/sec)	Carbon peak area (counts/sec)- double escape	Aromatic factor	Colloidal index
ARPECHIM	46,9	24,0	21,17	0.45
ESSO	93,33	30,94	13,4	0.23
EKO	96,07	13.45	3,31	0.29

In the first column are given the type of the sample. The final results were presented in fig. 3. The regression line can be described by an empirical relation.

$$C/H = 5.9 + 34.5 * I_C$$

with a correlation factor R = 0.99 and a standard deviation SD = 0.71.

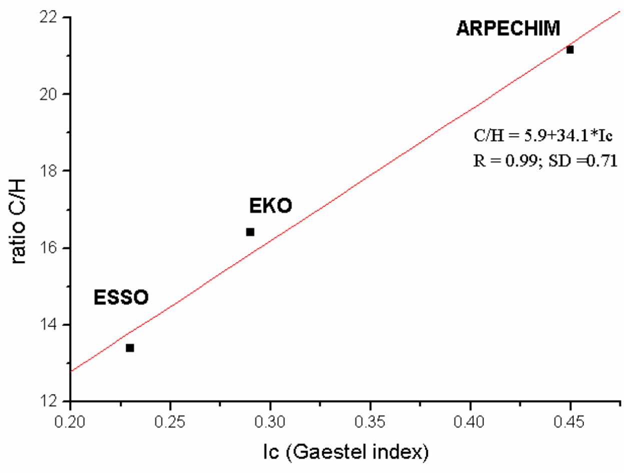

Fig.3. The aromatic factor C/H as a function of colloidal index.

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Considerations Concerning the Rehabilitation of Bridge Situated on 28 National Way km 6+950 over Siret River

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Abstract

National ways' modernization is a priority in the conditions of adherence process of Romania in European Union. Representative works are executed in Moldova in this acceptance, to the national ways 24 and 28.

Generally each rehabilitation solution imposes analyzes of routes: topographically, geodetically and of constructional solution.

The requirements of traffic fluency and circulation's security imposed expanding the width of tapes, leveling up new curves with higher rays, new bridges and after case, the rehabilitation of existing bridges.

The paper shows the constructional solution to rehabilitate the bridge situated at km 6+950, national way DN28, over Siret River and specifically topographical works to follow the bridge's comportment in exploitation conditions.

Keywords: *rehabilitation, follow comportment with geodetically method*

1. CONSOLIDATIONS WORKS

The bridge over Siret River situated at km 6+950 has a length of 236 m, four openings of 50.35 m (marginal opening) and 60.00 m (central openings). It has an infrastructure built by reinforced concrete and a metallic superstructure (Photo 1).

Because the landslides of foundation' terrain, at P3 pier could be observed an inclination of 40 cm in the river bed direction (Sãbãoani direction). This value was registered at the superior part of pier which had a tendency of loosing its stability. Was observed too the ruin of correspondent anchorage apparatus. In these conditions, before rehabilitation, National Agency of Ways had disposed a temporary wooden anchorage and the monitoring the pier inclination (Photo 2).

Rehabilitation's constructional works were realized during 2002, May and 2003, November, the authors' contributions being in monitoring these works and in following pier P3's compartment using geodetically specific methods in conditions of exploitation with and without velocity restriction.

Photo 1

The rehabilitation of P3 pier had included the following activities and constructional works:

- filling by natural stones of 50...400 kg/unit around the pier from the spread footing quota (photo3);
- banking the left side river (photo 4);
- gauging the river bed 290 m upstream and 300 m downstream and the protection of left side of river against water's erosion processes;

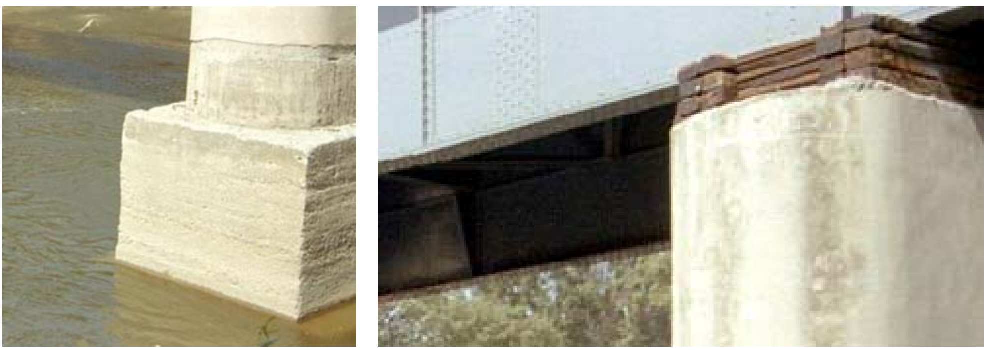

Photo 2

The defending the left river side was done in the same time in order to lead river's water in the opening named P3 between P2 and P3 pier. Locally near P2 pier was executed a temporary dam in order to protect it. (photo 5);

Photo 3

Photo 4

- Shirting pier P3 with 40 cm steel-concrete liner plate;
- fetching at the normal quota the bridge seat banquet and changing the anchorage apparatuses
- leveling up the superstructure at designed position (photo 6)
- replacing two destroyed bars of the metallic superstructure in area of leaning against pier P3.

Photo 5

Photo 6

2. FOLLOWING THE PIER'S P3 COMPORTMENT USING GEODETICALLY METHODS

In order to appreciate consolidated pier's comportment had considered a brisk networked formed by a fix points from national geodetic network (R_1), the station point (S_1) situated on the same line on the lateral bridge used for crossing pipes over Siret River, and supplementary, another 3 fix points situated on an imaginary line (C, D, E) having parallel direction against bridge's axis

For the new fix points C, D, E, the coordinates were obtained from a local map, in Stereo70 projection system.

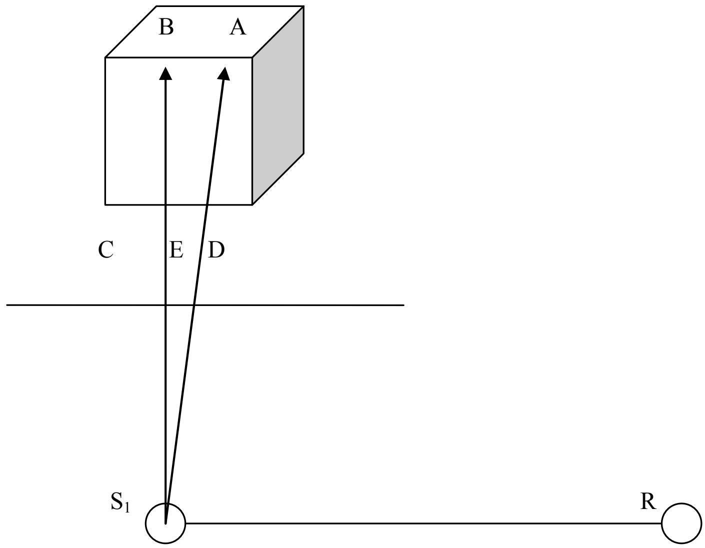

Figure.1

Values of coordinates were registered by total station, directly with a precision of 0.01 mm.

Measurements were executed in two series; Ist series: 7 values for point A, 10 values for point B and in IInd series: 17 values for point A, 12 values for point B. Values are shown in Table .

The first series is for rehabilitated bridge when vehicles had a velocity restriction of 20 km/h; the second is for the situation after rehabilitation, without velocity restriction for vehicles.

It be mentioned that the medium squared error in the station point is done by relation (1);

$$S_{\Delta Z} = \pm \frac{S_{\Delta Z}}{\sqrt{2}} = \pm \frac{S_C}{\sqrt{2}} = 0{,}08mm \qquad (1)$$

It can be observed from coordinates that the displacement of P3 pier was reduced after rehabilitation to those admitted in specialty literature. Practically points A and B are on the same line, parallel with bridge's axis.

Table 1

Visa’s Name	H_Z	V	ND	SD	dH	E	N	H	For id	Obs.’ Nr.
0	1	2	3	4	5	6	7	8	9	10
S_1-R_1	0.0005	116.0160	19.735	20.376	-3.697	1000.000	1019.735	96.303	A1555	366
S_1-R_1	0.0010	116.0160	19.734	20.376	-3.697	1000.000	1019.734	96.303	A1556	367
S_1-R_1	0.0010	116.0160	19.734	20.376	-3.697	1000.000	1019.734	96.303	A1557	368
S_1-R_1	399.9995	116.0160	19.735	20.376	-3.697	1000.000	1019.735	96.303	A1558	369
S_1-A	301.5880	103.2510	28.190	28.226	-0.066	971.819	1000.703	99.934	A1559	370
S_1-A	301.5865	103.2515	28.189	28.226	-0.066	971.819	1000.702	99.934	A1560	371
S_1-A	301.6110	103.2450	28.231	28.268	-0.065	971.778	1000.714	99.935	A1561	372
S_1-A	301.6115	103.2475	28.208	28.243	-0.065	971.802	1000.714	99.935	A1562	373
S_1-A	301.6095	103.2470	28.209	28.245	-0.065	971.800	1000.713	99.935	A1563	374
S_1-A	301.6095	10.32470	28.209	28.246	-0.065	971.800	1000.713	99.935	A1564	375
S_1-A	301.6095	103.2470	28.211	28.247	-0.065	971.798	1000.713	99.935	A1565	376
S_1-B	300.6555	103.2360	28.310	28.347	-0.065	971.691	1000.291	99.935	A1566	377
S_1-B	300.6555	103.2365	28.262	28.319	-0.064	971.719	1000.291	99.936	A1565	378
S_1-B	300.6530	103.2390	28.267	28.303	-0.064	971.735	1000.291	99.290	A1566	379
S_1-B	300.6530	103.2390	28.267	28.303	-0.064	971.735	1000.290	99.290	A1567	380
S_1-B	300.6555	103.2365	28.262	28.298	-0.064	971.400	1000.290	99.936	A1568	381
S_1-B	300.6530	103.2390	28.267	28.303	-0.064	971.734	1000.290	99.936	A1569	382
S_1-B	300.6530	103.2390	28.267	28.303	-0.064	971.737	1000.290	99.936	A1570	383
S_1-B	300.6535	103.2350	28.259	28.296	-0.062	971.742	1000.290	99.938	A1573	384
S_1-B	300.6545	103.2375	28.140	28.176	-0.057	971.862	1000.289	99.943	A1574	385
S_1-B	300.6545	103.2455	28.116	28.152	-0.059	971.886	1000.289	99.941	A1575	386
S_1-A	301.5915	103.2455	28.257	28.294	-0.067	971.752	1000.706	99.933	A1576	387
S_1-A	301.6115	103.2470	28.232	28.269	-0.066	971.777	1000.715	99.934	A1577	388
S_1-A	301.6160	103.2475	28.220	28.257	-0.066	971.789	1000.716	99.934	A1578	389
S_1-A	301.6180	103.2420	28.239	28.276	-0.066	971.770	1000.718	99.934	A1579	390
S_1-A	301.6175	103.2470	28.235	28.272	-0.066	971.774	1000.717	99.934	A1580	391
S_1-A	301.6150	103.2480	28.222	28.258	-0.066	971.787	1000.716	99.934	A1581	392
S_1-A	301.6100	103.2475	28.225	28.262	-0.066	971.784	1000.714	99.934	A1582	393

S_1-A	301.6100	103.2475	28.224	28.260	-0.066	971.785	1000.714	99.934	A1583	394
S_1-A	301.6095	103.2475	28.224	28.261	-0.066	971.785	1000.714	99.934	A1584	395
S_1-A	301.6100	103.2475	28.224	28.261	-0.066	971.785	1000.714	99.934	A1585	396
S_1-A	301.6095	103.2475	28.224	28.261	-0.066	971.785	1000.714	99.934	A1586	397
S_1-A	301.6100	103.2470	28.241	28.278	-0.067	971.783	1000.714	99.933	A1587	398
S_1-A	301.6100	103.2475	28.240	28.276	-0.067	971.769	1000.714	99.933	A1588	399
S_1-A	301.6100	103.2475	28.243	28.280	-0.067	971.766	1000.714	99.933	A1589	400
S_1-A	301.6105	103.2475	28.243	28.280	-0.067	971.766	1000.714	99.933	A1590	401
S_1-A	301.6100	103.2470	28.242	28.279	-0.067	971.767	1000.714	99.933	A1591	402
S_1-A	301.6100	103.2475	28.243	28.280	-0.067	971.766	1000.714	99.933	A1592	403
S_1-B	300.6510	103.2395	28.350	28.386	-0.069	971.652	1000.290	99.931	A1593	404
S_1-B	300.6535	103.2460	28.118	28.154	-0.060	971.884	1000.289	99.940	A1594	405
S_1-B	300.6535	103.2460	28.120	28.156	-0.060	971.882	1000.289	99.940	A1595	406
S_1-B	300.6535	103.2460	28.121	28.157	-0.061	971.881	1000.289	99.940	A1596	407
S_1-B	300.6530	103.2460	28.122	28.158	-0.060	971.880	1000.289	99.940	A1597	408
S_1-B	300.6530	103.2465	28.119	28.156	-0.060	971.882	1000.288	99.940	A1598	409
S_1-B	300.6530	103.2450	28.143	28.179	-0.061	971.859	1000.289	99.938	A1599	410
S_1-B	300.6530	103.2460	28.125	28.161	-0.060	971.877	1000.289	99.940	A1600	411
S_1-B	300.6530	103.2460	28.124	28.161	-0.060	971.879	1000.288	99.940	A1601	412
S_1-B	300.6530	103.2460	28.124	28.161	-0.060	971.877	1000.288	99.940	A1602	413
S_1-B	300.6530	103.2460	28.127	28.164	-0.060	971.874	1000.289	99.940	A1603	414
S_1-B	300.6530	103.2460	28.131	28.168	-0.061	971.870	1000.289	99.939	A1604	415
S_1-C	296.7950	120.1480	26.693	28.088	-7.367	973.341	998.657	92.633	A1605	416
S_1-C	296.7950	120.1480	26.693	28.088	-7.367	973.341	998.657	92.633	A1606	417
S_1-D	305.3580	120.0900	26.862	28.257	-7.395	973.233	1002.258	92.605	A1607	418
S_1-D	305.3580	120.0900	26.863	28.258	-7.395	973.232	1002.258	92.605	A1608	419
S_1-D	305.3580	120.0900	26.860	28.256	-7.395	973.235	1002.258	92.605	A1609	420
S_1-D	305.3625	120.0900	26.863	28.258	-7.395	973.232	1002.258	92.605	A1610	421
S_1-D	305.3615	120.0900	26.863	28.259	-7.395	973.232	1002.260	92.605	A1611	422
S_1-E	302.2680	120.1255	26.726	28.119	-7.367	973.291	1002.952	92.633	A1612	423
S_1-E	302.2670	120.1255	26.730	28.123	-7.368	973.287	1002.952	92.632	A1613	424
S_1-E	302.2670	120.1255	26.728	28.121	-7.368	973.289	1002.951	92.632	A1614	425

3. CONCLUSIONS

Rehabilitation’s works must be surveyed in their execution phase by a land surveying engineer, fact mentioned in Romanian quality law.
Generally, geodetically works carried on in the same time with constructional works could ensure a high quality results.
Constructional works must be protected against natural disaster like those produced by water erosion. In this purpose, it’s necessary to verify periodically the situation of stabilized slopes. Slopes’ sliding phenomena can affect the constructions situated in their vicinity.

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Numerical Method to Determine the Influence Surfaces of the Deck Bridges by Finite Strip

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Abstract

The aim of this paper is the establishing influence surfaces decks bridges as rectangular, oblique and trapeze modeling by isotropic and orthotropic plates with various boundary conditions, by using the method of finite strips technique.

The numerical method presents in the paper has the advantage of a low number of strips, which leads to a small number of equations and small width of the matrix band and it is an alternative to the numerical FEM and DFM.

Keywords: plate, strip, bridge deck, finite strip method, matrix, influence surfaces.

INTRODUCTION

The present paper is dealing with the Finite Strip Method associated with Rayleigh–Ritz` variational principle for determination the influence surfaces of the polygonal plates with varying boundary conditions. The method presented here is an alternative to the numerical FEM and DFM and brings more benefits as compared to these methods, both in the case of static and dynamic analysis, owing to the quick convergence of the solutions.

THE THEORETICAL PRESENTATION OF THE PROCEDURE

The plate is considered to be modeled as an assembly of narrow strips (assimilated to straight bars) parallel to the y axis. This finite strip technique respects the boundary conditions of the plate along both directions.

If the plate does not have a regular form the strips model the plate only approximately. The accuracy of the results is conditioned by the number of strips;

the greater the number, the better the accuracy. Plates of constant or variable thickness may be analyzed by means of this method under condition that the thickness of the strip is constant. The thickness may vary from a strip to the other.

The length of the strips may be constant (for rectangular or oblique plates) or variable (for trapezoidal or triangular plates).

Based on the energy principles of structural mechanics, a conservative material system acted by a conservative system of forces is in equilibrium if the total potential energy is stationary with respect the boundary conditions of the system.

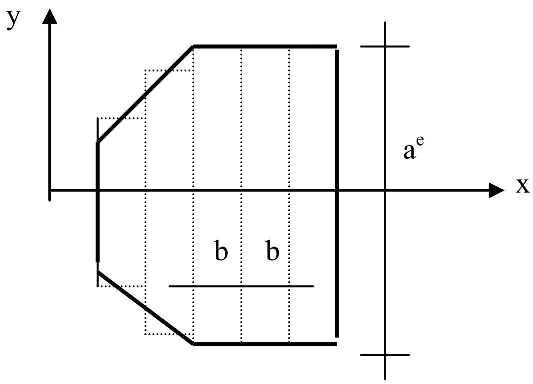


Figure 1. Mesh model of polygonal plates

$$V = U - L \tag{1}$$

V – the total potential energy
U – the strain energy of the plate
L – the work of external forces
d - generalised displacementes

Following [1], for one strip the total potential energy is

$$V^{(e)} = U^{(e)} - (L_P^e - L_q^e)$$

where
$L_P^{(e)}$ - is the work done by point loads
$L_q^{(e)}$ - is the work done by distributed loads

The strain energy of a strip has the following form [1]:

$$U^e = -\frac{1}{2}\iint (\varkappa^e)^T \mathbf{D} (\varkappa^e)\, dx\, dy \tag{2}$$

$\varkappa^e$ – is the matrix of curvature and has the following shape

$$\varkappa^e = \begin{vmatrix} -w_{,11} \\ -w_{,22} \\ 2w_{,12} \end{vmatrix} \tag{3}$$

and **D** is the elasticity matrix and has the following shape:

$$D = \begin{vmatrix} D_{11} & \upsilon D_{22} & 0 \\ \upsilon D_{22} & D_{22} & 0 \\ 0 & 0 & D_{12} \end{vmatrix}$$

$$D_{11} = E_1h^3/12(1-\upsilon_1\upsilon_2) \qquad D_{22} = E_2h^3/12(1-\upsilon_1\upsilon_2)$$

$$D_{12} = Gh^3/12(1-\upsilon_1\upsilon_2) \tag{4}$$

For the isotropic plate one has $\upsilon_1 = \upsilon_2 = \upsilon$ and $D_{11} = D_{22} = D = Eh^3/12(1-\upsilon^2)$ and $D_{12} = D(1-\upsilon)/2$.

Applying Ritz`s variational principle, the expression of the displacement is :

$$w^{(e)} = \sum_1^n \sum_1^m d^e_{ij}.\Phi_i(x) . F_j(y) \tag{5}$$

where :

Φ_i(x) – functions chosen to ensure the compatibility of linear displacement and rotations field across nodal lines

F_j(y) - functions chosen to ensure the edge conditions of the strip (Rayleigh functions)

d^e_{ij} - the generalised desplacements of the $e^{th.}$ strip

The expression number (5) may be written in a matrix form as:

$$w^{(e)} = \mathbf{C}.\mathbf{d}^e = [\ \mathbf{\Phi} F_1\ \ \mathbf{\Phi} F_2\ \ \ldots \mathbf{\Phi} F_n] \begin{vmatrix} \mathbf{d_1^e} \\ \mathbf{d_2^e} \\ \cdot \\ \cdot \\ \cdot \\ \mathbf{d_n^e} \end{vmatrix} \tag{6}$$

where

$$\mathbf{\Phi} = [\Phi_1\ \ \Phi_2\ \ \ldots.\ \Phi_n] \tag{7}$$

$$\mathbf{d_{ji}}^e = [d_{j1}{}^e\ \ d_{j2}{}^e\ \ d_{j3}{}^e \ldots\ d_{jn/2}{}^e\, d_{jn/2+1}{}^e \ldots\ d_{jn}{}^e] \tag{8}$$

The first n/2 elements are generalized displacementes (rotations and displacements) associated with the border line on the left of the strip and the other n/2 elements are associated with the border line on the right of the strip .

By substituting the (6) and (5) relationships in (2) and taking into account that :

$$B_j{}^e = \begin{vmatrix} -\,(\mathbf{\Phi} F_{j)},_{11} \\ -\,(\mathbf{\Phi} F_{j)},_{22} \\ 2(\mathbf{\Phi} F_j),_{12} \end{vmatrix} \tag{9}$$

Now, the expression of the strip stiffness matrix can be written:

$$\mathbf{K_{jk}}^{(e)} = \iint (\mathbf{B}_j^e)^T\ \mathbf{D}\ \ (\mathbf{B}_j^e)\ dx\ dy \tag{10}$$

In an extended form, this expression may be written as :

$$\mathbf{K}^e = \begin{vmatrix} \mathbf{K_{11}^e} & \mathbf{K_{12}^e} & \ldots. & \mathbf{K_{1n}^e} \\ \mathbf{K_{21}^e} & \mathbf{K_{22}^e} & \ldots. & \mathbf{K_{2n}^e} \\ \ldots.. & \ldots. & \ldots. & \ldots.. \\ \mathbf{K_{m1}^e} & \mathbf{K_{m2}^e} & \ldots. & \mathbf{K_{mm}^e} \end{vmatrix} \tag{11}$$

The strip stiffnes matrix is symmetrical and has 4n order. The matrix ($\mathbf{K}^e$) can be partitioned into the following way :

$$\mathbf{K}^e = \begin{vmatrix} \mathbf{K_{SS}} & \mathbf{K_{SD}} \\ \mathbf{K_{DS}} & \mathbf{K_{DD}} \end{vmatrix}$$

The plate stiffness matrix is obtained by assembling the matrices given by (11) and is given by:

$$\mathbf{K} = \begin{vmatrix} \mathbf{K^1_{SS}} & \mathbf{K^1_{SD}} & & & & \mathbf{0} & & \\ \mathbf{K^1_{DS}} & \mathbf{K^1_{DD}+K^2_{SS}} & \mathbf{K^2_{SD}} & & & & & \\ & \mathbf{K^2_{DS}} & \mathbf{K^2_{DD}+K^3_{SS}} & \mathbf{K^3_{SD}} & & & & \\ & & \mathbf{K^3_{DS}} & \mathbf{K^3_{DD}+\ldots} & \cdot & & & \\ & & & & \cdot & & & \\ & & & & \cdot & & & \\ & \mathbf{0} & & & & & \mathbf{\ldots+K^r_{SS}} & \mathbf{K^r_{SD}} \\ & & & & & & \mathbf{K^r_{DS}} & \mathbf{K^r_{SS}} \end{vmatrix} \qquad (12)$$

where:

r – represents the order of the general matrix **K**. **K** is a band matrix, positively defined and having null elements except for those of the band.

$$r = 2n.(NEF + 1) \qquad (13)$$

n – the number of the functions of approximation

NEF – the number of the finite strips which the plate is divided into.

With these elements the expression of the strain energy for one strip of plate is:

$$U^e = \frac{1}{2}(\mathbf{d^e})^T \mathbf{K}^e (\mathbf{d^e}) \qquad (14)$$

and for the entire plate is equal to:

$$U = \sum_1^r U^e = \frac{1}{2}\mathbf{d^T K d} \qquad (15)$$

The work done by forces and distributed loads is given by

$$L_P^{(e)} = \sum_1^s P_i w_i^{(e)} = \sum_1^s P_i C^T(i)(d^{(e)})^T = \mathbf{S_P^e}(\mathbf{d^e})^\mathbf{T} \qquad (16)$$

$$L_q^{(e)} = \iint_{S^e} \mathbf{q w^e dxdy} = \iint_{S^e} \mathbf{qC^T (d^e)^T dxdy = S_q^e (d^e)^T}$$

With expressions (14) ,(15) and (16) the total potential energy is written as it follows:

$$V = U - L = \frac{1}{2} \mathbf{d^T Kd - (S_P + S_q) d^T} \qquad (17)$$

Considering the condition of stationarity of the total potential energy, with respect the generalized displacements, the expression (17) takes the form of the following matrical equation:

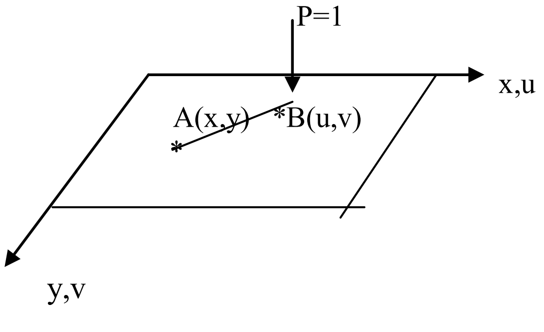
$$\mathbf{Kd - (S_P + S_q) = 0} \qquad (18)$$

and

$$\mathbf{d = K^{-1}(S_P + S_q)}$$

The displacement of a point, is obtained from (5) by using generalized displacements and the functions F_j and Φ_i.

The influence surfaces may be computed considering the mobile force P = 1acting in point B. The displacement w is computed and then it is represented in point A as in the figure bellow.



NUMERICAL RESULTS

A computer program has been elaborated in VISUAL BASIC, based on the theory presented above. This program computes the influence surfaces of the plates with several geometrical forms and several boundary conditions.

In what follows, the computation of the influence surfaces of the bending moment around y axis for a simply supported square plate is presented. The computed values are compared with those obtained by Olsen and Reinitzhuber in [5].

The expressions of the approximating functions from the relation (5) and used in this computation are:

$$F_j(y) = \sin(j\pi\, y/a)$$

$$\Phi_1(x) = 1-3x^2/b^2+2x^3/b^3$$
$$\Phi_2(x) = x-2x^2/b+x^3/b^2$$
$$\Phi_3(x) = 3x^2/b^2-2x^3/b^3$$
$$\Phi_4(x) = x^3/b^2-x^2/b$$

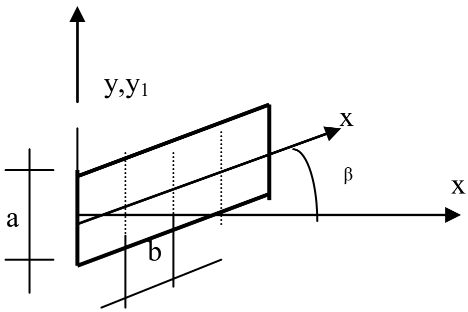


Figure 2 Finite strip mesh of the oblique plates

Tabel 1: Influence surface **S** m_y along the edge of the plate computed via FSM

	1	2	3	4	5	6	7	8	9
a	0.574	0.451	0.310	0.234	0.179	0.140	0.112	0.092	0.075
b	0.354	0.342	0.270	0.209	0.163	0.129	0.103	0.085	0.070
c	0.195	0.192	0.176	0.149	0.120	0.097	0.079	0.065	0.054
d	0.088	0.089	0.084	0.075	0.063	0.051	0.042	0.035	0.029

Tabel 2: Influence surface **S** m_y along the edge of the plate computed in [5]

	1	2	3	4	5	6	7	8	9
a		0.461	0.316	0.234	0.179	0.140	0.112	0.092	0.075
b	0.350	0.331	0.268	0.209	0.163	0.129	0.103	0.085	0.070
c	0.193	0.192	0.175	0.149	0.120	0.097	0.079	0.065	0.054
d	0.089	0.090	0.085	0.075	0.063	0.051	0.042	0.035	0.029

Tabel 3: Influence surface **S** m_y in the middle of the plate computed by FSM

	FSM $\nu = 0$					in [5] $\nu = 0$				
	1	2	3	4	5	1	2	3	4	5
a	0.179	0.196	0.223	0.270	0.277	0.179	0.197	0.224	0.277	
b	0.163	0.177	0.192	0.205	0.198	0.163	0.176	0.192	0.203	0.190
c	0.120	0.127	0.131	0.125	0.119	0.120	0.127	0.130	0.124	0.117
d	0.063	0.065	0.063	0.0592	0.0372	0.063	0.065	0.064	0.06	0.037

Input data:

$\beta = 0$

$\nu = 0$

The plate has been divided into 7 strips. The point where the y values have been computed are the same as those given in Olsen and Reinitzhuber to allow the comparison of the values given by the two methods.

CONCLUSIONS

The Numerical Method presented in the paper has the advantage of a low number of strips, which leads to a small number of equations and small width of the matrix band.

By using this method, the author developed computer programs to compute the influence surfaces of the isotropic and orthotropic polygonal plates with several boundary conditions.

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The dynamic and the impact of highway research and technology on social and economical development of Romania

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1. HISTORY AND TRADITION

There is a strong tradition, deeply anchored into its history for building roads and bridges in Romania. This country, geographically located north of the Danube River and on the Black Sea coast, in the South-East region of Europe, was part of the great Roman Empire. Its name of those times was Dacia Felix[1], and even today, vestiges of the old Roman roads and bridges[2] , which lasted over millennia, are witnessing this. Later on, during the 19-th century, the modern road and railway bridge, designed and constructed over the same Danube river at Cernavoda by the Romanian engineer Anghel Saligny was considered to be, at that time, one of the largest bridge in Europe. Nowadays, the tradition is continued with the modern bridges designed and constructed as part of the Romanian section of the Trans- European North-South Motorway (TEM) and with the actual significant road rehabilitation program[3], aiming the integration of the country's transportation infrastructure into the European's one.

2. THE ROMANIAN PUBLIC ROAD NETWORK.

According the official statistics [1], the public road network of Romania, has a total length of 198,589 Km. As mentioned in Table 1, the national road network has a total length of 14,696 km, from which 5,576 km are classified as European roads. This road network is characterized by a number of 3,271 bridges, with a total length of 136,683 m.

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[1] Dacia Felix – The Happy Dacia
[2] The remains of the Bridge over Danube, at Turnu Severin, designed and constructed by Apolodor of Damascus, under the order of Trajanus the Roman imperator, in 56 a.c.
[3] http://www.andnet.com

Table 1. The Romanian road network [1]

Motorways	114 km
National Roads (under the NAR's administration and management)	14.696 km
County Roads (under the administration of County Councils)	36.010 km
Communal Roads (under the administration of Local Councils)	27.781 km
Urban Streets	22.328 km
Rural Streets (under the administration of city, town and communes	97.660 km

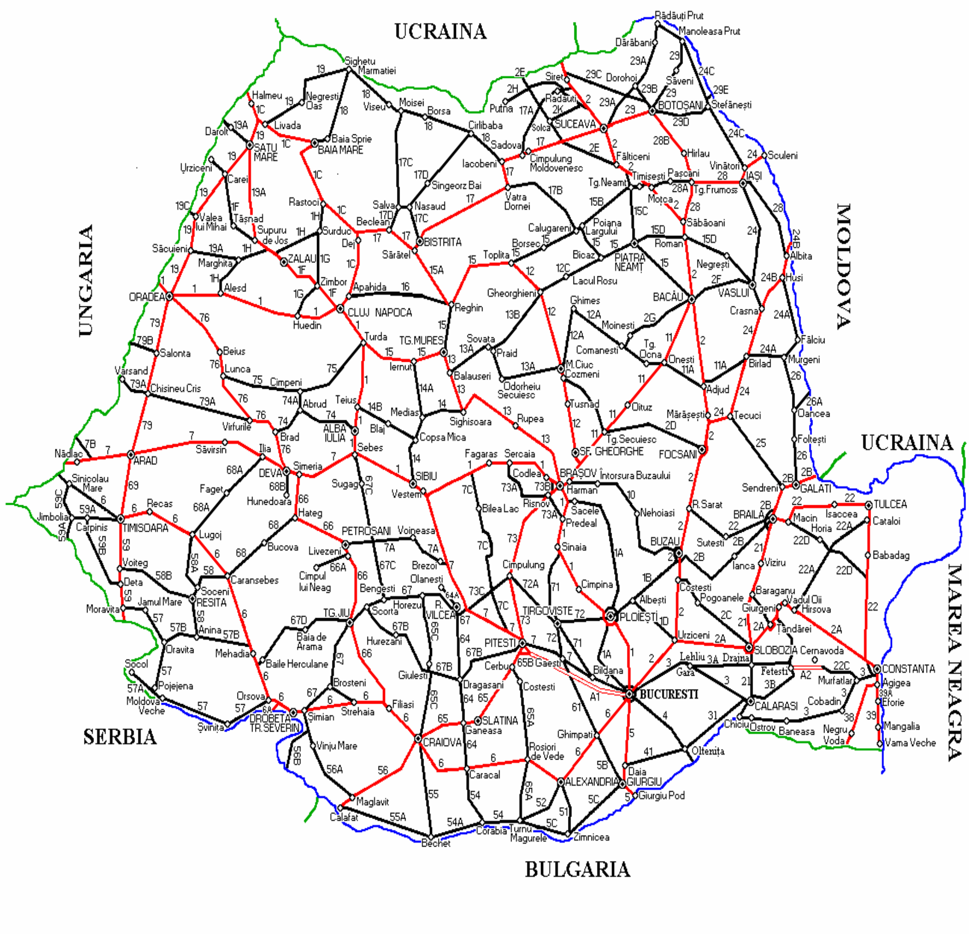

Fig.1. The map with the schematic national road network of Romania

Traffic on the Romanian road network, as shown in Fig.2, is characterized by an Annual Average Daily Traffic (AADT) of 4,531 physical vehicles with a percentage of heavy (commercial vehicles) of approximately 25% from the total volume of traffic. More then 85% of the national roads are provided with asphalt flexible pavements, only 13% of roads are equipped with cement concrete pavements and 2% are still gravel roads.

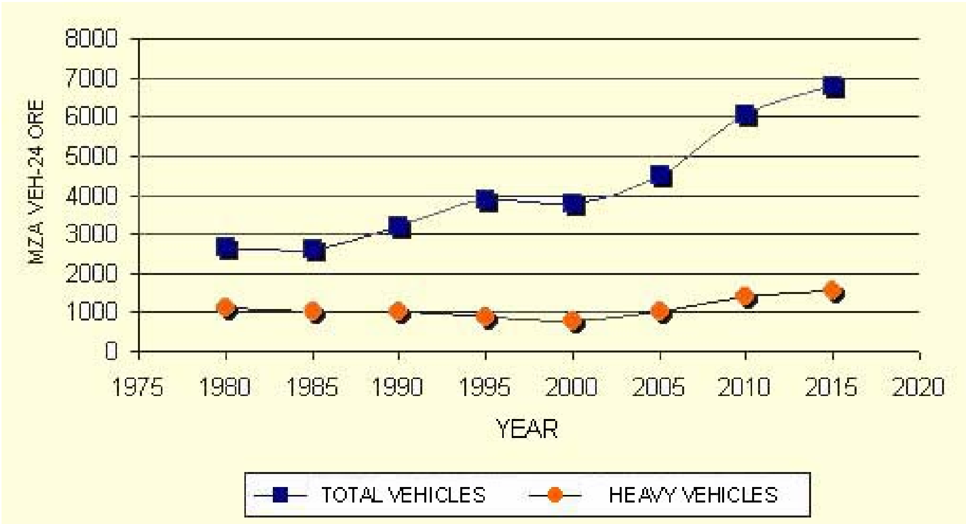

Fig.2. Evolution and forecasting of road traffic on the public road network of Romania [1]

3. RADICAL CHANGES. STEPS TOWARDS POLITICAL AND PROFESSIONAL DELIVERANCE

The radical changes , which took place in the political, economic social and cultural life after the 1989 anticommunist Revolution, had a significant impact on the highway policy of he country , the previous national strategy based mainly on strongly encouraged rail transport mode being rapidly reoriented towards the development of road transportation, which deliberately was kept for a long period of time at its lowest operational and serviceability limits. The National Administration of Roads which was recently transformed into a National Agency for Roads & Motorways S.A., has as its main objective the ensuring a homogeneous development of the entire national road network. **In this respect it participate in the various international road events and activities, and collaborates with European and other international bodies to bring the Romanian road strategy in line with the international regulations and conventions on the road infrastructure, road traffic, inter-modal transport and European transport corridors.** In this context, in parallel with the road network development, new road research strategies and policies have been adopted, with benefic impact on the social development of Romania. Some of these strategies and actions such as : reorientation of previous national transport strategy, participation of Romania with a loan staff to the developments of the Strategic Highway Research Program-SHRP; initiation and running of the Romanian Long Term Pavement Performance Program : RO-LTPP; establishment and running of a complete SHARP/SUPERPAVE Laboratory[2], and affiliation of this Center to the European Forum of the Highway Research Laboratories – FEHRL[3],[4]; education of Romanian young professionals for earning their master or doctoral degrees in prestigious universities activities from USA, South Africa and Europe and the recent developments towards the creation of a Virtual South-East European Center for Durable Pavements (Superpave) at the technical University “Gh. Asachi” of Iasi [5], could be considered as significant milestones of this dynamic .

4. ROMANIAN PARTICIPATION IN SHRP PROGRAM. THE ESTABLISHMENT AND RUNNING OF A SHRP/ SUPERPAVE LABORATORY IN BUCHAREST

As well as in other European countries, the performance based specifications for road bitumen, issued by the Strategic Highway Research Program (SHRP) have caused a significant impact on the Romanian research trends and also on road technology itself, now essentially oriented towards the Superpave approach.[2],[6] The attraction exerted on Romanian specialists of this approach, can be explained by both its originality and the fact that these new specifications are relatively distinct from the specifications in effect existing in Romania and in most European countries some years ago. Romania had the chance to participate with loan-staff in SHRP and our road specialists, confronted with critical problems, raised mainly by the poor quality of some indigenous bituminous binders, and law performance of their pavements under the traffic explosion occurred during the last year, have found the innovations brought by the new SHRP specifications, very useful in their endeavor to build better roads in this country.

The major innovations brought by SHRP specifications, and considered of great interest by the Romanian specialists are as follows:

(i) the classification system based on notion of climatic zones, each zone being defined by winter and summer temperature extremes;

(ii) the required performance levels (performance grade – PG) does not vary according to the grade, they remain constant, whereas the operating conditions vary as a function of the climatic zones;

(iii) SHRP tests are carried out not only on fresh bitumen as supplied by refineries, but also after the artificial aging of bitumen simulating short term (RTFOT) and respectively long term (PAV) aging;

(iv) The specifications are meant to apply not only to pure bituminous binders but also to the modified ones.

4.1. The Superpave Approach in Romania.

In its quality of technical research unit of National Road Administration, the Romanian Centre for Road Engineering Studies – CESTRIN, has taken since 1993 year, a particular interest in Superpave approach, focusing mainly on the quality of indigenous bitumen, intended to be used in the huge Road Rehabilitation Program involving over 3000 km road network undertaken by the Ministry of Transport.

A research strategy for short and long term have been established for developing a customized version of Superpave technology adapted to the specific traffic & climatic conditions of this country including procurement of SHRP test equipment for binder and mixes initiation of climatic research studies for recording and processing of air and pavement temperature distribution in Romania, and initiation of the Romanian Long Term Pavement Performance Program: RO-LTPP.

In this respect, since 1995 year, in the frame of the Romanian Center for Road Engineering Studies and Informatics, have been established , with the technical assistance of American specialists from Asphalt Advanced Technologies L.P. from US, a complete SHARP/Superpave Laboratory for testing asphalt binder and mixes on performance based specifications [2],[6].

This laboratory has proved to be very useful tool for the evaluation of the performance of all bitumen binders, supplied by various national and international companies during the last ten year and nowadays, and used for the construction of the Romanian sector of Trans-European North- South Motorway and for the actual road rehabilitation program , encompassing over 5,000 km of the existing national and European roads**.** Being the only functional Superpave laboratory in the south-east European region, it is now possible to fulfill also international requests for Superpave testing.

4.2. Development the climatic database and of a PG map, for Romania

In order to properly implement the Superpave technology in this country extensive research has been undertaken by CESTRIN, to develop an appropriate climatic database with the collaboration of the **Romanian Institute for Meteorology & Hydrology** witch provided the specific raw data, and with **the Universities of Iassy & Timisoara** [7],[8], for the processing of these data and their conversion into the pavement surface temperature, followed by the calculation of the maximum and respective minimum pavement temperatures at a specified depth. Example of such temperatures and the corresponding PG's are given in Table 2.

Table 2. The extreme temperature values of road pavements and their corresponding P.G. for some of the meteorological stations, situated in the East region of Romania [8], [9]

The meteo station	$T_{S(max)}$ °C	$T_{S(min)}$ °C	PG x-y
Bucharest/Baneasa	57.65	-9.5	PG 58-16
Grivita	57.33	-8.9	PG 58-16
Iasi	54.5	-11.6	PG 52-16
Suceava	52.1	-12.5	PG 52-16
Joseni	51.6	-28.9	PG 58-34

As the traffic conditions can increase the strains in the mass of road pavement, the P.G. determined only by climate, has to be adjusted, as function of traffic, as follows:

a) consideration of traffic volume, by converting the actual 11,5KN axle load to the 8 KN (ESAL):
- for a volume less 10^7 ESAL, no grade correction
- for a volume between 10^7 and 3 x 10^7 ESAL, possible increase of one high-temperature grade
- for a volume greater than 3 x 10^7 ESAL an increase of one high-temperature grade

b) consideration of traffic speed:
- for non moving vehicles, in parked areas, an increase of two high-temperature degree
- for slow-moving traffic, an increase of one high-temperature grade
- for fast-moving traffic, no grade correction

Finally the PG have been defined by departing from the mean and standard deviations for the maximum and minimum air temperature, for the confidence levels of 50% and 98% followed by their conversion into pavement temperature and the respective speed and traffic volumes correction. For example, at the confidence levels chosen, the grade for Joseni [see Table 1] is PG 46-28 (50%) or PG 52-34 (98%).

4.3. The performance – grade evaluation of indigenous bituminous binders. Research for improvement quality of binder, in SHRP terms

Based on Van der Poel's diagram [10] which permits the evaluation of the stiffness modulus from the Ring & Ball temperature, penetration index, temperature and time of loading, the main parameters of some indigenous bituminous binders, specified by Superpave, have been estimated. These estimations have been confirmed by appropriate SHRP tests and in this respect from extensive studies performed in our laboratory and abroad it has been found that common bituminous binders grades 60/80, (associated with performance grades PG of 64°C), and the common grades 80/100 (associated with PG 58°C), cannot be considered effective in responding to heavy or intense traffic. In order improve their behavior, at high temperature and also to accommodate pavements to the increasing severity of traffic conditions, research using various inert or reactive modifiers have been undertaken since 1995 year, in CESTRIN Laboratory and various international laboratories such as LCPC in France [11], Advanced Asphalt Technologies, Du Pont Polymers in USA, etc.. This improvement obtained in performance by modification is much better illustrated by the suggestive five-axis spider diagram from Fig.3, which plots the same SHRP test results.

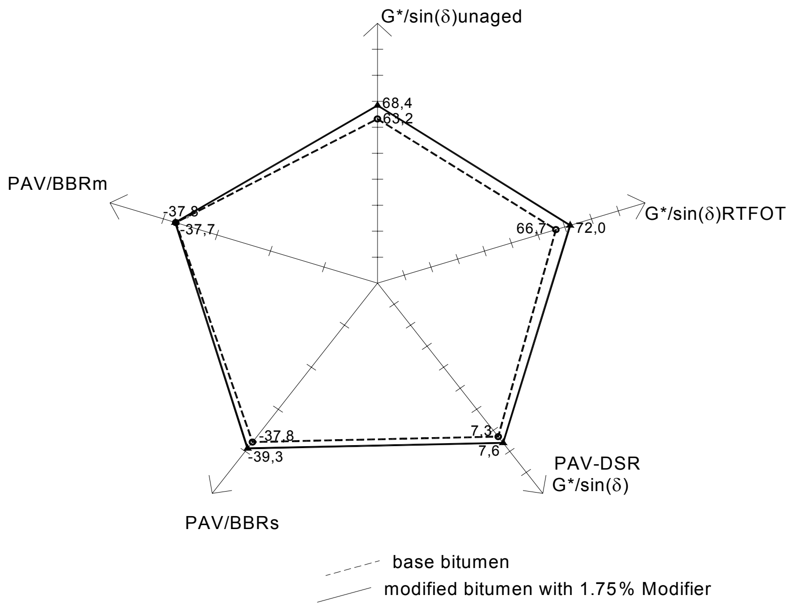

Fig.3. SHRP test results on base bitumen and on the same bitumen, modified with 1,75% percent reactive polymer(ELVALOY AM) [12],[13]

For other indigenous bitumen (ASTRA) it was found that the improvement obtained by modification, is even higher, the original PG increasing after modification with two grades.

Similar research and SHRP screening have also been performed in NRA/CESTRIN on imported bitumen from various sources (Spain, Greece, USA, Italy, etc.) determining their performance, in order to accept or reject their use in road rehabilitation or current works, the SHRP specification becoming thus an important tool for technical evaluation.

On may conclude that in the frame of the research undertaken by NRA/CESTRIN on various indigenous or imported bituminous binders, the SHRP specification based on rheologic parameters, determined under specific climatic and traffic conditions of this country, became a very important tool for the evaluation of the performance of these binders and also to quantify the effect of various quality improvement techniques.

Also the initiation and future development of the pavement temperature & PG map are now permitting to determine the PG of any road site project in Romania. One of the main conclusions drawn from these early research results is that in the specific climatic conditions of this country (continental climate with severe winters & very hot summers) the base indigenous or imported ones do not satisfy the SHRP specifications, and there is a great need to improve them, by using adhesive agents and appropriate modifiers.

In this respect it was found that in order to get a better performance of the base bitumen it is necessary to use for modification at least three inert polymers, as follows:

- a first one, to improve performance, at very low temperatures, to prevent fatigue and thermal cracking;
- a second polymer to improve performance at high temperature, to prevent rutting;
- a third one, to improve the homogeneity of the bituminous – polymer blend, and to prevent their separation.

This approach leads to very sophisticated technology difficult to be managed in the site, the performance of pavements realized with such polymers, in various climatic zones of Romania, being very poor.

The actual trend is focusing now the use of the base bitumen improved with reactive polymers [12],[13], which are cheaper, more compatible with most types of base bitumen, this eliminating the sophisticated installations for modification of bitumen, and greatly simplifying the technology.

The SUPERPAVE mix design – level one, is now implementing in order to check the various types of mixes, used in road rehabilitation works. Thus mixes of SMA types have been recently adopted in Romania [14],[15] under the name of bituminous mixes stabilized with fibers, MBSF 16 or MBSF 8 using various binders improved with appropriate additives or / and modifiers in order to address the severe climatic and traffic conditions, specific for this country.

The strategy adopted ten years ago shall be continued in the years to come, in order to fully validate the Superpave technology, parallel with the harmonization of the national specifications for road works with the European standards and trends.

5. THE ROMANIAN RO-LTPP PROGRAM. DEVELOPMENT OF SPECIFIC ROAD RESEARCH USING THE ACCELERATED TESTING FACILITY FROM TECHNICAL UNIVERSITY " GH. ASACHI" OF IASI IN CONJUNCTION WITH THE RESEARCH DONE ON THE RO-LTPP SECTORS

In 1992/1993 Romania was one of a few East European countries participating with a loan staff to the developments of the Strategic Highway Research Program – SHRP and inspired from this , at the end of SHRP, it has been initiated the actual Romanian Long Term Pavement Performance Program: RO-LTPP, with the active implication of the Technical Universities of Iasi, Timisoara, Bucharest and Cluj-Napoca. This successful program [15] includes more then fifty representative LTPP sectors , selected on the entire public rod network . A first five year report results have been used for the development of specific distress equations for the actual Pavement Management System. Other early research results from RO-LTPP program was used to improve the actual structural methods for design, in conjunction with the results obtained on the Accelerated Testing Facility ALT-LIRA[4] (Fig 4), from technical University "Gh. Asachi" of Iasi.[16].

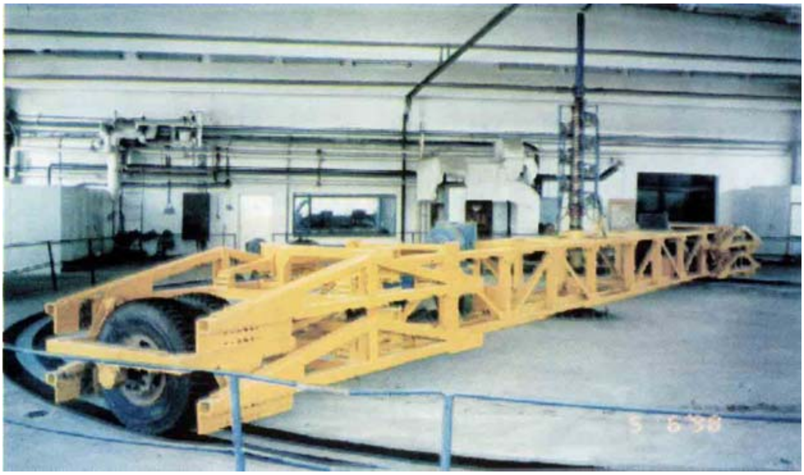

Fig.4. General view of the Circular ALT Facility- LIRA from technical University " Gh. Asachi" of Iasi [16]

Thus Since the year 1993, marking the beginning of a huge and resolute effort of National Administration of Roads, directed towards the rehabilitation the public road network, the Romanian specialists have been confronted with the difficult task

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[4] ALT- LIRA : Accelerated Loading Tests facility from Technical University "Gh. Asachi" of Iasi

of selection and implementation of new asphalt technologies, in order to replace the old and outdated ones, and to permit the design and construction of stronger and better flexible pavements. These new pavements were seek to exhibit a better performance of the existing road network, to the severe traffic and climatic conditions , characterized by the sudden increase of the traffic volume, parallel with the adoption of the axle load of 115KN, and by huge temperature gradients between the hot and cold seasons.

A first and successful step realized in the frame of this strategy, was the research and implementation in the current road rehabilitation practice of the MASF16/8 type mixes [14], [15],[16] stabilized with various fibers, customized to the specific technical properties if the Romanian aggregates and binders. The application of these superior mixes is now generalized on all road rehabilitation projects, in this country.

A second step, was the adoption and implementation of the specific testing technology [17] for assessing the susceptibility of these mixes to rutting, in the conditions of very high temperatures reached in the asphalt pavement during the summers, for some regions, these temperatures over passing 65^0C, according SHRP Algorithm [8].

A third and very important step [16] was the undertaking, in conjunction with related Ro-LTPP sectors, of accelerated testing of these type of mixes, stabilized with indigenous or imported fibers, in order to assess and validate, in a short time, their behavior and performance under the specific new adopted axle load of 115 kN. Thus the results of a last five years research study undertaken in the frame of Accelerated Load Testing (ALT) facility at the Technical University "Gh. Asachi" of Iassy, brought and continues to bring significant contributions to the performance evaluation and validation of technical specifications for the asphalt mixes stabilized with various fibers, used for the construction of bituminous road pavements in the frame of the huge ongoing effort of road rehabilitation in Romania.

6. AFFILIATION OF THE NATIONAL CENTER FOR ROAD ENGINEERING STUDIES TO FEHRL

The Forum of European Highway Research Laboratories (FEHRL)[3], [4], initially created on 1989 year by directors of 13 European countries, with the purpose to encourage collaborative research between the member laboratories in the field of highway engineering infrastructure the is now comprising the national road research laboratories in all member sates of European Union (EU) as well the European of Free Trade Association with affiliate members in another seven East European countries from the former Warsaw Pact, after

Poland and Slovenia , Romania becoming,, through its Center for Road Engineering Studies and Informatics -CESTRIN , in 1995 year , one of the new first affiliate member , and then, in 1999 getting the status of an affiliate-founding member of this important organization. Taking this opportunity , in pursuing the objective of research cooperation and using the cooperation mechanisms employed by FEHRL, Romania became actively involved in the various levels of the association (FEHRL Board, FEHRL Research Coordinators, etc), and bringing significant contributions to the development of some of the COST[5] programs such as COST 345 -Procedures Required for Assessing Highway Structures, COST 347 -Accelerated Load Testing for Road Pavements, COST 350 Integrated Assessment of Environmental Impact of Traffic and Transportation Infrastructure, or being involved in other FEHRL projects, thus gaining an increasing recognition in the European road sector. Recently, in May 2003, the FEHRL Executive Board of Directors , hold its periodical meeting in Bucharest, hosted by the National Administration of Roads (NAR[6]), this event opening new opportunities for further implication of Romanian specialists into the European activities.

7. TOWARDS NATO AND EUROPEAN INTEGRATION. RECENT UNDERTAKINGS FOR CREATION OF A VIRTUAL SOUTH-EAST EUROPEAN FORUM /CENTER FOR SUPERIOR PAVEMENTS, ASSOCIATED WITH TECHNICAL UNIVERSITY “GH. ASACHI” OF IASI

As mentioned above, in 1992, Romania began active participation in the Strategic highway research Program (SHRP). Initially this participation was carried out through the provision of loaned staff from the Romanian Center for Road Engineering Studies and Informatics. During the last twelve years, Romania has maintained closed contacts with TRB, FHWA and with American technical Universities engaged in research and deployment of Superpave (R) system, including University of Maryland, Arizona State University and LSU. Contacts also have been maintained with American consulting engineering firms such as Asphalt Advanced Technologies Inc., in order to transfer and implement in Romania the various outputs of the SHRP and LTPP programs. One of the main outputs of this cooperation was the initiation, in 1992, of the Romanian –LTPP program within the framework of the National Administration of Roads and under the management of CESTRIN and with the involvement of all four civil

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[5] COST- Cooperation in Science and Technology/Cooperation Scientifique et Technique
[6] NAR- National Administration of Roads now re-organized under the name of National Company for Roads and Infrastructure S.A.

engineering universities in Romania. Another significant achievement was the procurement by the Romanian NAR of the complete set of SHRP equipment for testing asphalt binder and mixes and the implementation of a customized version of SHRP/ Superpave technology. This procurement was made with World Bank funds and with the participation of American specialists from asphalt advanced technologies, Inc., for the procurement, installation and training of the Romanian specialists on Superpave issues. At this stage Romania posses the most complete set of laboratory equipment for testing binder and mixes according SHRP/Superpave technology in central and Eastern Europe. These circumstances provide the opportunity for the creation of a SHRP/Superpave/LTPP Virtual Center for the South-East European region, to be hosted by Technical University “Gh. Asachi” of Iasi in cooperation with NAR/ CESTRIN in Bucharest. Such a Center having as main objective the improvement the technical cooperation in the region, constant provision of aid in the dissemination of knowledge about hot-mix asphalt pavement design and construction. The Center could also provide training and assistance to professionals from the region. This opportunity was specifically recognized, approved, and included in the Final resolution of the IRF International Road Congress for south east Europe, held on 23-25 October 2001 in Bucharest, Romania. Responsible executives from other European countries expressed, in the framework of recent FEHRL meetings, their interest in Superpave technology, and indicated they would send their specialists to become familiarized and be trained in Romania’s Center. Very recently, in year 2003, the Bulgarian specialists were the first from the Region , who applied and benefited for performing tests and assistance from Technical University of Iassy and Cestrin. Taking into consideration this opportunity, Technical University “Gh. Asachi” of Iassy wants to move forward with the creation of this virtual Center , similar with those found in the United States, and in this respect , as this incipient stage, was seeking supplementary assistance , to turn this opportunity into reality. In this respect, pursuing the resolution of IRF Regional Congress[7] , held in Bucharest on 2001 year, some concrete undertakings have been undertaken in order to explore the possibility to establish in Romania of a regional Superpave Center , with the technical assistance of TRB and FHWA. Almost a complete SHRP/Superpave library has been provided in this respect by the TRB and some of SHRP /LTPP and Superpave Documents have been translated and adapted for implementation in the everyday practice . Thus at this stage the following SHRP and LTPP documents have been translated and customized to the specific Romanian traffic , climatic and geographical conditions : “ SHRP/ LTPP Guide for Distress Investigation”, “LTPP Guide for Investigations], “ SHRP Algorithm”, “ SHRP Climatic Data Base”, etc.

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[7] IRF- International Road Federation Regional Congress

The first idea was to establish the headquarter of this center in NAR/CESTRIN, in Bucharest, where is now located the SHRP Laboratory, but now, with a more active involvement in national road research of the Technical Universities and in the light of the recent talks held with the TRB and FHWA representatives, on January 2004 in Washington D.C., the decision is to organize this headquarter like a Virtual Superpave & Infrastructure Knowledge Center at the Technical University "Gh. Asachi" of Iasi and to cooperate in this respect with the other three technical Universities of Romania and with CESTRIN /SHRP Laboratories. Besides the existing technical information received from TRB, the following needs, as discussed with the TRB representatives, has to be fulfilled , in order to make this Center functional:

- To complete the existing set of SHRP/ Superpave reports with the other SHRP /LTPP products for the actual library of the Center, such as LTPP Data Pave , last version , SHRP /SUPERPAVE software and other computer programs;
- To complete with appropriate SHRP/Superpave products for display at a mobile, mini-exhibition to be regularly housed at the Center headquarter, and elsewhere, as needed;
- To get technical assistance and expertise for the review and assessment of the current capabilities of the center;
- to get technical expertise and assistance to review and advise upon the development of the main activities of the Center , such as regional validation and calibration program using selected road sectors of the Romanian LTPP program, training, forensic investigations, etc;
- To get advice and assistance for the finalization of a customized version of SHRP/ Superpave technology for Romania and for the Southeast European countries;
- To get advice on the establishment of a permanent training scheme on SHRP/ Superpave and other LTPP issues for the professionals and practitioners from the region. such advice is likely to include a needs assessment, curriculum development, and identification of suitable teaching or training materials found elsewhere . Technical University "Gh. Asachi" of Iasi will assume responsibility for translation and adaptation.
- To get assistance in promoting the SHRP activities and products into the academic environment by introducing SHRP issues in the current University curriculum, probably through training of selected Romanian professors in US. These selected professors would train counterparts in Romania.

So it is envisaged that in the near future, through the Civil Engineering Universities from Romania, in the frame of the Center, will be possible to implement a continuing education system on SHRP/ Superpave subjects, this system being already fully developed in some universities. A first teaching material, drafted under the form of a university training course [18] has been published by the Technical University of Bucharest and is still in use since the year 2001.

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Draft methodology for assessment of road pavement condition, based on the combined concept of chance, change and entropy

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Summary

This research report presents a new methodology for the assessment of road pavement condition, based on three powerful concepts, aimed to accommodate the uncertainty caused by the complex interrelationships, material defects, structural deficiencies, human errors, and ambient fluctuations characterizing the pavement systems. These very useful concepts, selected with the aim, to approach, in a new comprehensive manner, the difficult task of assessment of road pavement condition are as follows: the concept of chance, expressed in terms of probabilities and reliability; the concept of change, expressed in terms of dynamics, e.g. of change of the pavement condition in time; the concept of entropy, expressed in terms of differences in pavement system's entropy, occurred in time, as an appropriate quantitative parameter, measuring the degree of uncertainty of a pavement to meet its specific structural, surface and traffic safety requirements. This paper does not intend to present the methodology of extracting the stochastic information from laboratory or from the routine pavement measurements and the assignment of single-valued point estimates, these techniques being, at this stage , well known in the highway engineering media, and accurately described in the existing literature, and assumes that such estimates and also their inherent variability expressed in terms of tendencies, standard deviations and coefficient of correlation, obtained by tractable probabilistic analyses, for correlated random variables, governing the performance of road pavements, are just known, and that they can be used, as raw material data for the new proposed methodology.

KEYWORDS: pavement condition, chance, probability, change, dynamics, entropy

1. BACKGROUND

There is a growing awareness of the road maintenance problem in the actual world. Many countries are incurring extremely high costs because of inadequate maintenance of their road network. Lack of effective maintenance is leading to the need for premature rehabilitation of their roads and is causing high costs for vehicle operation, for industry and for agriculture. In most countries the maintenance budget is inadequate and consequently, road administrations have to spend their budgets effectively and in such a way as to achieve the best value for the money. However, constraints of available resources make it necessary to set priorities for budget allocation, and these are needed at all levels in the hierarchy, from national to local level. That's why the accurate assessment of the road condition, in order to establish the maintenance needs and to set such priorities, is very important. There are a great variety of methods for the assessment of the road condition (Robinson,1986); some of them are based on visual assessment, while others combine the visual assessment for ranking roads according to their condition with physical measurement surveys (deflections, roughness, distress, etc.), at sample points to enable the maintenance needs to be determined. In some cases the results of visual assessments are combined with such physical measurements using statistical relationships, which are also based on subjective appraisal. All these methods involve a certain amount of uncertainty and are less repeatable. Also, the actual interpretation of these data, in order to obtain an integrated parameter with a unit value to express the global condition of the road pavement, is not adequate. Our research (Andrei, R., 1992, 2002) has been oriented towards the development of a method for the quantitative evaluation of the road condition, by converting the various measured parameters, such as deflections, roughness, skidding resistance, rutting and distress index, etc., each of them being expressed in specific units, into a unique, integrating parameter, capable of expressing more precisely, the global road condition. This research report presents a new methodology for the assessment of road pavement condition, based on three powerful concepts, aimed to accommodate the uncertainty caused by the complex interrelationships, material defects, structural deficiencies, human errors, and ambient fluctuations characterizing the pavement systems. These very useful concepts, selected with the aim, to approach, in a new comprehensive manner, the difficult task of assessment of road pavement condition are as follows:

- The concept of CHANCE, expressed in terms of probabilities and reliability;

- The concept of CHANGE, expressed in terms of dynamics, e.g. of change of the pavement condition in time;

- The concept of ENTROPY, expressed in terms of differences in pavement system's entropy, occurred in time, as an appropriate quantitative parameter, measuring the degree of uncertainty of a pavement to meet its specific structural, surface and traffic safety requirements.

This paper does not intend to present the methodology of extracting the stochastic information from laboratory or from the routine pavement measurements and the assignment of single-valued point estimates, these techniques being, at this stage , well known in the highway engineering media, and accurately described in the existing literature (Harr.E. M., 1984; Harr E.M.,1991), and assumes that such estimates and also their inherent variability expressed in terms of tendencies, standard deviations and coefficient of correlation, obtained by tractable probabilistic analyses, for correlated random variables, governing the performance of road pavements, are just known, and that they can be used, as raw material data for the new proposed methodology. The advantage of this new approach, in comparison with the exiting ones, consists mainly in the elimination of the numerous weighing coefficients, empirically determined and widely used in the old methods. Also, the new method uses simple dynamic graphical representations of the specific road conditions. According the degree of complexity involved in the analyses of the road pavement condition, these dynamic graphical representation, derived from the well known concept of change, can be developed under the form of diagrams (e.g. for the evaluation of the road pavement condition of a large road network),trigrams, tetra grams or even hexagrams (e.g. for the evaluation of the road pavement condition at a local, regional level, where the decisions taken have to be more specific, in terms of the maintenance works necessary to perform, in order to bring the pavements to the technical conditions required by the environmental and traffic conditions). According this new approach, to any of the possible configuration expressing a specific road pavement condition, existing at a given moment during its design life, it is associated an identification number, determined in terms of topological entropy, this number being very useful, first for the identification of the real case encountered in practice and second, for the establishing of the limits of priority classes for the rehabilitation works. Any line in these diagrams, trigrams or hexagrams, represents a specific road pavement condition, such as bearing capacity of the pavement structure, roughness, skidding resistance, rutting, etc., any of these conditions being expressed in terms of entropy, as sole criterion to rank the relative uncertainty of the pavement to meet the specified requirements.

This global approach, has been possible to be undertaken, due to the continuous progress of the scientific knowledge of the road pavement behavior, registered in the last years (Harr, 1991), and with the advent of the last developments in the field of modeling natural systems (Casti, J.L., 1989), entropy optimization (Kapour, 1993) etc.

2. PRINCIPLES OF MODELING NATURAL SYSTEMS APPLIED TO THE ROAD PAVEMENTS

Lets consider S as a given road pavement structure , which usually has a finite number of sets of abstract space states W, characterizing its distinct physical states, defined as follows:

$$W = \{w1, w2, w3,...\} \qquad [2.1.]$$

In accordance with the theory of the modeling of the natural systems (Casti,1989), an observable of S is a rule f associating a real number with each w belonging to S , and this could be written as follows:

$$f : W - R \qquad [2.2.]$$

where R is the multitude of the real numbers.

Let assume that in a particular case, from the simplest point of view concerning its bearing capacity, a road pavement structure S is characterized by a set W of four abstract states: w1, w2, w3 and w4, defined as follows:

w1 - the difference between the specific probability values (the probability level, p1, for which the measured bearing capacity will have to meet the technical specifications required for that pavement, and the minimum probability value, characterizing its complete failure, p0=0), for which the maintenance organization has to do nothing, concerning the improving the existing bearing capacity of the pavement;

w2 - the difference between the specific probability values, defined as for the previous state, for which some maintenance works, such as asphalt surface treatments, has to be performed by the maintenance organization, in order to preserve the existent bearing capacity of the pavement;

w3 - the difference between the specific probabilities, defined as for the previous states, for which rehabilitation works, such as construction of asphalt or concrete overlays, has to be performed by the maintenance organization, in order to improve the existing bearing capacity of the pavement;

w4 - the difference between the specific probabilities, defined as for the previous states, for which, the maintenance organization has to decide and perform the complete reconstruction of the existing pavement, in order to bring it to the technical parameters, required by the specific traffic and environmental conditions;

In this particular case we may define the observable of the system S, f1 : W - R , by the rule:

f1(w) = difference between the specific probabilities

or f(wi)=pi-p0

where i=1,2,3,4

If, for example, we consider the specific probability levels: p1=0.96; p2=0.76; p3=0.56 and p4=0.37, the set W of observable f1(w) will be as follows:

f1(w1)=p1-p0=0.96

$f1(w2)=p2-p0=0.76$

$f1(w3)=p3-p0=0.56$

$f1(w4)=p4-p0=0.37$

One may observed, that the system S, can also be described by the observable f2 : W - R, with a different rule:

f2(w)=difference between the specific risks of failure of road pavement, or:

$$f2(wi)=qi-q0$$

in the particular case considered above, the set W of observable f2(w) being as follows:

$$f2(w1)=q1-q0=0.04-1.00=-0.96$$

$$f2(w2)=q2-q0=0.024-1.00=-0.76$$

$$f2(w3)=q3-q0=0.44-1.00=-0.56$$

$$f2(w4)=q4-q0=0.63-1.00=-0.37$$

The system S can also be described by the set W of observable f3: W - S, by a rule involving the difference between its specific entropies, as follows:

$$f3(wi)=-pi*\log pi - (-p0*\log p0)$$

In our particular case the set W of the observable f3(w), should be:

$$f3(w1)= -p1*\log p1- (-p0*\log p0) =0.056537$$

$$f3(w2)= -p2*\log p2- (-po*\log p0) =0.300905$$

$$f3(w3)=- p3*\log p3- (-p0*\log p0) =0.468440$$

$$f3(w4)= -p4*\log p4- (-p0*\log p0) =0.530729$$

Let consider again these observables, defined as above:

f1(w1)=0.96	f2(w1)=-0.96	f3(w1)=0.055537
f1(w2)=0.76	f2(w2)=-0.76	f3(w2)=0.300905
f1(w3)=0.56	f2(w3)=-0.56	f3(w3)=0.46488
f1(w4)=0.37	f2(w4)=-0.37	f3(w4)=0.530729

The question is, which of these observables (f1, f2, f3, ..., fn) gives us the possibility to see more of the system S, and note also the linkage relationships between the observables f1, f2 and f3, these relationships being, as follows:

$f1(wi)=-f2(wi)$

$f3(wi)=-f1(wi)*(\log f1(wi))/\log2$

$$f1(wi)=-f3(wi)/((\log f1(wi))/\log 2)$$

$$f2(wi)= f3(wi)/((\log f1(wi))/\log 2)$$

One may observe, that although the observable f2 alone, contains less information about the system S, than the observables f1 or f3 , this lack of information can be compensate for if we know the linkage relations, mentioned above. But this is just an example. Generally speaking, in order to "see" the complete system S, we would need an infinite number of observables fi : W - R, so we can conclude finally that the complete system S is described by W and the entire set of observables F = {fi}. For practical reasons, in the particular case of the road pavement system, it's inconvenient to work with such large set of observables, so we have to just throw most of them away and to focus our attention on a proper subset of F, this subset being in fact, an abstraction of S. Thus, the real problem in our particular case is to find a good abstraction, more useful and practical one, and in order to find it we can use different techniques, even mathematical tricks and subterfuges, all these aimed at finding the best abstraction, capable to describe more completely the road pavement condition. Finally, we may conclude that a specific road pavement condition consists of an abstract state space W, together with a finite set of observables:

$$fi : W - R, \quad i=1, 2, 3, ...,m$$

Symbolically, our road pavement, considered as a natural system S, can be written as follows:

$$S = \{W, f1, f2, ...,fm\} \qquad [2.3.]$$

the observables {f1, f2, ...,fm}, providing the raw data by which we finally, can see its global condition.

The set of relationships linking these observables forms the equation of state for a specific road pavement, and formally it can be written as follows:

$$Qi(f1, f2,...,fm) = 0 \quad i=1, 2, ...,m \qquad [2.4.]$$

In our study we shall follow, intuitively this general and abstract approach, each of the observables f1, f2, ...,fm, aiming to describe a particular road pavement condition, in terms of relative uncertainty (entropy), as will be shown in Chapter 5.

3. THE CONCEPT OF CHANCE

Both definitions of the concept of chance, e.g. the definition based on subjective probability and that one based on relative frequency, are more useful in road pavement engineering. The relative frequency, defines the probability of an outcome event E, P(E), as the ratio between the number n of the outcomes favorable to the event E and the total number N of possible outcomes, and it is used when one has to deal with processes which are repeated a very large number of times. This definition can be written as follows:

$$P(E)=n/N \qquad [3.1.]$$

In the particular case of global evaluation of the condition of a road pavement structure, when the concept of repeated processes become meaningless, we are using the so called subjective probability, defined as the ability of the pavement to be successful or to fail in meeting the specific technical requirements established by the road administration, according to the environmental and traffic conditions affecting that pavement. According the existing literature (Harr,1984), this definition can be written as follows:

$$P(\text{Success \& Failure}) = 1 \qquad [3.2]$$

The probability of success of a pavement structure is called reliability, R, and its probability of failure p(f),is also called probability of risk, and with these symbols, the relation 3.2. defining the subjective probability, becomes :

$$R + p(f) = 1 \qquad [3.3]$$

Generally speaking, the adequacy of a road pavement to meet its technical conditions may be determined, by comparing its actual level of performance with that specified one. The specified level of performance will be a function resultant of many uncertain components of the pavement structure under consideration, such as traffic loading, location of the underground water table, climate and temperature, etc.. In the same time, the actual level of performance of the pavement structure will be also a resultant function which will depend on the variability of material parameters, testing errors, construction and supervision procedures, ambient conditions, etc. The ratio between the mean value of the safety margin, sf, and its standard deviation, sd, is called reliability index (Harr, 1984). Practically, the reliability index distribution is unknown, and this may reduce considerably its adequacy to express or to predict the performance of a road pavement, but despite this aspect (Harr, 1987), it may be used as a basis for evaluation of the performance of many civil engineering structures, as well as in conjunction with the relative entropy concept in order to rank the performance of road pavements.

4. THE COMBINED CONCEPTS OF CHANCE AND CHANGE APPLIED TO THE ROAD PAVEMENTS. DYNAMICS AND SPECIFIC PATTERNS, EXPRESSING THE GLOBAL ROAD PAVEMENT CONDITION AND ITS TRENDS

From the ancient times, the human mind has been preoccupied with the chance aspect of events, and a great amount of human effort has been directed for preventing, combating or restricting the danger, represented by chance.Our modern science is based upon the principle of causality and is considered to be an axiomatic truth, but very often these axioms of causality are being shaken to their foundations, and with the progress of modern physics, we finally understood that what we call natural laws are merely statistical truth, every natural process being affected by chance, so that, under natural circumstances, a course of events absolutely conforming to specific laws is almost un exception (Young, 1990).

In the struggle of the human mind to handle the events of chance, the matter of interest has been from the very beginning, the specific configurations formed by chance events in the moment of observation, and than the global interpretation of these configurations. This concept may be extended to the particular case of road pavements, subjected to the specific traffic and climate conditions, where different types of events of distress, caused by the variability of the pavement materials, may occur simultaneously, determining a specific pattern of the pavement condition. Such a pattern is, in fact, a specific configuration of the effects of these various distress events, whose development and evolution in time are very often governed by the laws of chance. This configuration of distress events, governed by chance, and forming a specific pattern at a given moment is a dynamic pattern, permanently changing his structure with time.

From the time of Confucius and with the ancient Greeks, the man contemplated the thesis expressing this idea of change and the modern mathematical manifestations of this concept, stating that " all things are in flux ", are resumed under the general notion of the dynamic system (Casti, 1989).

If we consider the road pavement condition as being a dynamic process, providing a great variety of behavioral patterns, the complex problem of determining the road pavement condition at a given moment during its design life will consists in identifying its specific pattern, related to that moment of observation, based on the results obtained by performing the measurement of the various parameters, each of these parameters, expressing a specific aspect of the global pavement condition. After the identification of the appropriate pattern from the multitude of the possible existing patterns, the next step may consist in selecting a convenient strategy from the multitude of strategies which can be priory developed as parts of the pavement management system adopted. Also, even the specific laws of different types of distresses are less known, by applying the principles of the optimization of entropy (Kapour, 19992) one may assign an appropriate probability distribution, according the range of information available (Harr, 1987) , e.g. the Uniform distribution when we don't know anything about the real distribution, ; the exponential law, when we know at least the expected value of the distribution; the Normal distribution when we know the expected value and the range of the standard deviation, etc. As it has recently been demonstrated, (Kapour, 1992) the most standard probability distributions, such as the uniform, geometric, exponential, gamma, normal, beta and Cauchy distributions, can be obtained by maximizing the classical, Shannon entropy measure.

The use of this principle may be of particular importance for the processing and interpretation of the LTTP data available at this stage, even the real probability distributions of the evolution of the different parameters involved it is not well known. According to the principles of modeling natural systems described earlier (chapter 2), if we assume that our road pavement system S, is described by a set of abstract states X, defined as follows:

$$W = \{w1,w2,...,wn)$$

then the dynamic on W is defined as a rule, specifying how a given state w transforms to another state Tt(w) at a time t, this rule being written:

$$Tt(x) : W - W$$

where, Tt is called "the transformation", and both the state set W and the time set Tt can be either continuous or discrete variables. This rule describes " a flow" or "a change" of state of the dynamic system W. Knowing or assuming ,in terms of entropy, the dynamic rule of state-transition and the initial state w0 W, than we should easily find answers to the questions, related with our system, such as follows:

-what kind of pattern will emerge in the long term limit?

-how fast these limiting patterns are approached during the design life of a specific road pavement?

-which are the types of the initial states (patterns) giving rise to different classes of limiting patterns?

Ultimately, all these questions, are coming down to the fundamental question of weather or not certain types of limited behavior are possible under this dynamic rule. As an example of such a state-transition process, lets consider the sequence of states which may occur in a road pavement structure, in time, every of these states (e.g. x1= structural condition; x2= surface condition; x3=safety condition),developing in time according a specific pattern (law), so that the transition - matrix of the pavement condition from the initial state to the states 1, 2, 3, can be written as follows:

		P (next state)		
		1	2	3
	1	p11	p12	p13
P (initial state)	2	p21	p22	p23
	3	p31	p31	p33

The elements of this matrix, pij, designate the probability that the initial condition i will become state j in the next transition.

5. THE CONCEPT OF ENTROPY. ENTROPY OPTIMIZATION PRINCIPLES. DIFFERENT MEASURES OF ENTROPY. TOPOLOGICAL ENTROPY AND ITS USE FOR PATTERN IDENTIFICATION

Any new road is supposed to be constructed to specification requirements with at least a 95% confidence level, and may be considered as a complex macroscopic physical system, characterized by a high degree of order, such a road being, in fact, without any significant defaults. After its completion and opening to traffic, coming under the combined incidence of the repeated actions of the wheel loads, of climatic and other environmental factors, etc., the road will suffer a certain decrease of its initial degree of order. As we know, in order to measure the uncertainty of a physical system, a special notion, (Shannon, 1948; Ventsel, 1973) has been developed. This notion, derived indirectly by application of the second principle of Thermodynamics into Informatics, is called <u>entropy</u>. Let us consider the physical system (S), and its global condition W, at a given moment, characterized by a finite number of state conditions: w1, w2, ... ,wn, the probabilities related to these conditions being: p1, p2, p3,... , pn. For convenience, these data may be arranged as follows:

$$\frac{(wi):\ w1\ \ w2\ \ w3\ \ \ldots\ \ wn}{(pi):\ p1\ \ p2\ \ p3\ \ \ldots\ \ pn}$$

In this case the value of the <u>entropy</u> of this system, H(S), shall be determined with the relation :

$$H(S) = -\ \ pi*\log pi \qquad (5.1.)$$

According literature (Ventsel, 1973), in order to establish a quantitative unit measure for the <u>entropy</u> we may consider the most simple physical system, X, that of tossing a coin, which presents only two equally probable conditions, p1 = p2 = 1/2. By applying relation 5.1. to this particular case, one obtains :

$$H(X) = -\ [(1/2*\log 1/2) + (1/2*\log 1/2)] = 1$$

The <u>entropy</u> unit obtained in this way has been called a <u>bit</u>, (binary digit). The evaluation of the entropy of a physical system, by using the relation (5.1.), may be simplified by introducing a special function (fi), defined with the relation (5.2.):

$$(f) = -p*\log p \qquad (5.2.)$$

and in this case, the relation (5.1.) becomes (5.3.):

$$H(S) = \ \ fi \qquad (5.3.)$$

the function fi being tabulated, (Ventsel, 1973) for various values of probability, between 0 and 1.

If we consider more complex systems, composed from a finite number of independent sub-systems, the entropy of the entire system will be determined by summing the partial entropies, using a relation similar to the relation (5.4.), developed for the case of a complex system (Z), composed of two independent sub-systems (X) and (Y):

$$H(Z) = H(X,Y) = H(X) + H(Y) \qquad (5.4.)$$

Usually, the sub-systems forming a complex physical system are dependant upon each other, and in such a case the relation (5.4.) developed for independent sub-systems is no longer applicable. In such a case, conditional entropy applies. Thus, the entropy of the entire complex system (Q), composed of three interdependent sub-systems: X, Y, and Z shall be determined by using the relation (5.5.):

$$H(Q) = H(X,Y,Z) = H(X) + H(Y|X) + H(Z|X,Y) \qquad (5.5.)$$

where the conditional entropy of each sub-system is calculated in the hypothesis that the conditions of all previous sub-systems are known. Now, considering the road pavement condition as a complex system composed, e.g., of five main sub-systems, characterized by the various parameters such as roughness, distress, bearing capacity, skidding and comfort, the maximum value of the road entropy, Hmax, calculated in the hypothesis that all these sub-systems are independent, shall be as follows:

$$H_{max} = 5 * h_{max} = 5 * 0.5307 \text{ bit} = 2.6535 \text{ bit}$$

and for practical convenience, this value may also be expressed in milibit units, so:

$$H_{max} = 2653.5 \text{ mbit.}$$

Let consider an examples of the evaluation of the specific road pavement condition, in terms of relative uncertainty, e.g. ,in terms of specific entropy. Given the following deviations, hi, of elevations from a straight line, at points of 3.00 meter intervals, along the traveled lanes of five roads, we should try to rank the roughness of the pavements, in terms of uncertainty (entropy):

Deviation	Road :				
hi(mm)	a	b	c	d	e
h1	12	13	11	9	15
h2	18	15	15	12	14
h3	16	12	19	10	18
h4	16	13	21	9	21
h5	13	15	8	10	20

h6	12	19	10	7	19
h7	19	16	12	11	7
h8	10	10	20	12	10
h9	16	11	18	10	9
h10	10	14	13	8	7

Calculations of the conventional statistics (average, standard deviation, Coefficient of variation) and of the classical (Shannon) and maximum entropies (Max. Ent.) are given in the Appendix...,(the worksheet Roughness1.Wk1). The following synthetic data are extracted from this worksheet:

Road:	a	b	c	d	e
Sum(mm)	142	138	153	98	140
Avg.(mm)	25.818	25.090	27.818	17.818	25.454
STD.(mm)	3.059	2.481	4.051	1.536	5.157
Dif.Ent.(byte)	0.338	0.023	0.052	0.018	0.102

Analyzing these data, one may conclude that, from the all five roads considered, the flattest is the road "d", its entropy being very near the maximum entropy; the roughest is the road "a", for which the difference between its real entropy and its desired entropy (Max.Ent) is maximal.

A similar approach, based on entropy, may be used in ranking road pavements according their structural or safety conditions. In most general cases, where the global road condition is considered as a complex system composed from independent and interdependent sub-systems, the entropy value characterizing the whole road shall be calculated by using a relation similar to the relation (5.5.) above.

6. THE USE OF THE COMBINED CONCEPTS OF CHANCE, CHANGE AND ENTROPY FOR EVALUATION OF ROAD PAVEMENT CONDITION AND RANKING THE MAINTENANCE AND REHABILITATION WORKS.

In order to develop an adequate methodology for the evaluation of the partial and global condition of an existing road pavement, at a given moment of its life, let us consider the simplest pattern, involving its structural condition, noted as ,"xxx" and its surface condition, noted as "yyy".

This general pattern can be represented graphically as a diagram, as follows:

xxx
*
yyy

Further on, in order to simplify things, we assume that each of these pavement conditions is divided into different classes of behavior, each class being characterized by a specific probability matrix, as follows:

Parameter	Symbol	Probability to meet technical conditions specified for the road pavement: pmin	Pmax.
aaa)	**Structural condition:**		
x	xxx	0.76	0.99
x1	x1x	0.56	0.75
x2	x2x	0.37	0.55
x3	x3x	0.01	0.36
bbb) Surface condition:			
y	yyy	0.76	0.99
y1	y1y	0.56	0.75
y2	y2y	0.37	0.55
y3	y3y	0.01	0.36

From these classes of behavior, we can develop the matrix of the all possible situations, which may occur in practice, as follows:

	x	**x1**	**x2**	**x3**
y	x,y	x1,y	x2,y	x3,y
y1	x,y1	x1,y1	x2,y1	x3,y1
y2	x,y2	x1,y2	x2,y2	x3,y2
y3	x,y3	x1,y3	x2,y3	x3,y3

The graphical representations of these sixteen possible diagram patterns are shown below:

	xxx	x1x	x2x	x3x
yyy	xxx * yyy	x1x * yyy	x2x * yyy	x3x * yyy
y1y	xxx * y1y	x1x * y1y	x2x * y1y	x3x * y1y
y2y	xxx * y2y	x1x * y2y	x2x * y2y	x3x * y2y
y3y	xxx * y3y	x1x * y3y	x2x * y3y	x3x * y3y

Any of these patterns has its specific probability matrix, as for example the pattern no.1, as follows:

Pattern1:		Probability limits	
		pmin.	Pmax.
Graphical symbol:	xxx *	0.76	0.99
	yyy	0.76	0.99

From the multitude measures of entropy developed and used in different fields of research, the following have been selected and computed in our study ,with the aim to find out which one is more suitable and advantageous for the specific case of evaluation of the road pavement condition:

a) the classical entropy (Shannon, 1948), described as above and determined with the relation:

$$H(pi) = - \quad pi*\ln(pi)$$

b) the generalized measure of entropy (Kapour, 1992), determined with the relation:

$$- \ pi*\ln(pi) \ - \quad (1-pi)*\ln(1-pi)$$

c) the topological entropy

Topological entropy is more important for the identification of different patterns of road pavement condition. The topological entropy is defined as a measure of the likelihood of particular configuration (Casti,1988). We can also define spatial measure entropy, formed from the probabilities of possible sequences, and may occur in the frame of a specific configuration and also temporal entropy to account the number of sequences that may occur in time -series of entropy values taken by each subsystem of a given configuration. Topological entropies reflect the all possible configuration of a system; measure spatial/temporal entropies reflect those configurations which are more probable. In order to define more precisely a sole road pavement condition, expressed by a specific configuration, it is often necessary to connect up the topological and the measure spatial/temporal entropies, according with the complexity of the system. If we have N, a sequence of numbers defining a specific road pavement condition configuration, with k parameters describing its global condition, the topological entropy, Ht, reflecting this specific configuration, will be calculated with the relation;

$$Ht = 1/N*\log \quad (N)$$

the total number of possible sequences n, will be :

$$n = K\hat{}i$$

where i is the number of possible cases of any individual condition.

As usually, the number of possible cases varies with each individual condition, and in such situations, the evaluation of the number of all possible sequences, n, will have to take in account these variations.

7. CONCLUSIONS AND FURTHER RECOMMENDATIONS FOR THE DEVELOPMENT AND USE OF THIS APPROACH, FOR DESIGNING MORE EFFICIENT PAVEMENT MANAGEMENT SYSTEMS

1. This paper presents the results of a research study conducted with the aim of investigating, in a detailed manner, the possibility to develop a new methodology for the evaluation of road pavement condition.

2. This new methodology is aimed to be based on the combined use of three powerful concepts, mainly:

-the concept of chance, expressed in terms of probabilities;

-the concept of change, expressed in terms of dynamics and transformations;

-the concept of entropy, expressed in terms of uncertainty or disorder, and measured in specific informatics units (bits)

3. The study was not intended to present the routine methodology for the extraction the stochastic information from laboratory or from routine pavement investigations, assuming that such techniques are well known, and their results, expressed in terms of tendencies, standard deviations or coefficients of correlation, obtained by tractable probabilistic analysis, may be used as raw data for the new proposed methodology.

4. With the purpose of simplifying the calculations and for conducting the study in a more systematic manner, the multitude of the parameters, influencing the global road pavement condition, have been grouped into the following main classes:

- parameters of structural condition (mainly, bearing capacity)

- parameters of surface condition (mainly: roughness, rutting, distress)

- parameters of safety condition (mainly, skidding resistance)

- parameters of comfort condition (mainly, the relation traffic/geometric design; landscape, environment)

5. For the first time the comfort condition as described above, was intended to be introduced as an essential part of the global pavement condition, in the future developments of the Pavement Management Systems.

6. In this initial stage, the study has been developed only for the simplest pattern of pavement condition, represented under the form of a diagram, including two main conditions: e.g. the structural condition (xxx) and the surface condition (yyy), as shown below:

xxx

*

yyy

7. According with the degree of complexity involved in the analyses of specific road pavements , the patterns used can be also under the form of trigrams, tetra grams or even hexagrams. Thus ,the simple patterns such a s diagrams ,trigrams or tetra grams may be used for the assessment of pavement condition for huge road networks, at national or federal levels ,while the hexagram patterns can be

developed for a more detailed assessment at local or regional road network levels. The research results, developed for the study of the simple patterns(diagrams) could be extended "mutatis mutandis" to the more complex patterns, the number of the possible existing patterns increasing exponentially from sixteen ,as in the particular case of the diagrams, to some thousands in the case of the more complex patterns.

8. Further on, each of the four main conditions mentioned above has been divided into three to four sub conditions, defined each one, according its specific probability matrix. For a more suggestive graphical representations of these patterns , the following symbols have been used for the different lines forming a specific pattern: xxx or ---- (continuous line) , representing a sub condition which generally meets the technical requirements, so that there is no need to undertake any important maintenance or rehabilitation action; x1x or -- -- (yielding line), having in its middle space a cipher (1,2 or 3), representing the gravity of the condition and consequently the maintenance or rehabilitation measure which are recommended to be undertaken, in order to bring the pavement to its specified requirements, e.g., as follows:

1- need for routine maintenance measures

2- need for overlay or special maintenance actions

3- need for reconstruction or radical maintenance measures.

9. The specific study of the diagram patterns,, have been conducted with the aim to find out, from the different measures of entropy, used nowadays in different research fields, the most suitable ones for the specific study of the road pavement condition. The results of the study showed that the classical(Shannon)measure of entropy is following a similar symmetry law as well as the probability measures, giving the same figure for symmetrical patterns of behavior (e.g. ;patterns: xxx/y1y & x1x/yyy). However, the classical measures of entropy may be used successfully, for the evaluation of an individual parameter (e.g. roughness, etc) , in order to rank the different roads on the sole criterion of entropy, which is more powerful than other criteria such as standard deviation, coefficient of variation or reliability, as it is shown in the example given in the Chapter 5.

10. Also, from the research results have been found that more important than the usual measures of entropy are the differences between these measures and their corresponding maximum entropies, obtained by applying the principles of maximization of entropy (Max.Ent) , as it is shown in the examples given in the Appendix, in relation with the patterns 1 & 2

11. It appears from this research, that the principle of optimization of entropy (Max. Ent.) should be of particular interest for the processing and interpretation of the existing LTTP data , by taking advantage of the fact, that even if the probability distribution of a specific parameter is not yet known in its totality (as it

is often the case in the LTTP data), the principles of Max. Ent. may assume according the degree of uncertainty of a specific case, a typical standard distribution, in order to assess or to predict the future evolution of a specific parameter.(e.g. if the probability distribution is completely unknown, on may assume in our study the uniform distribution, also if only the expected value of the distribution as known, on may assume the normal distribution, etc..)

12. In order to eliminate the disadvantages caused by the symmetry of the patterns occurring in the case of classical measures of entropy, the topological measure of entropy has been investigated and it was proved to be very useful for the purpose of the individual pattern identification from the multitude of the possible existing patterns of road pavement condition.

13. During the documentation study on the subject of entropy related with the assessment of pavement condition, , it was very interesting to find out that in a research study ,concerning road pavement performance , conducted by the University of Alabama, in which 31 experts rated the performance of 1086 road sections, on a scale of 0 to 100, the probability value for which they recommended major rehabilitation ,P=0.379 was very near the probability P=0.37, for which one may get the maximum value of entropy (Max. Ent.) Finally, taking into consideration the results of these research and the advantages offered by the various entropy measures in characterizing partially or globally the road pavement condition, we strongly recommend further developments , at a greater scale ,in terms of time, teams for work and complexity of patterns considered, and coordinate study to be undertaken into the future in the frame of research LTPP program, and also the use of the results of this study in the analysis of the existing LTPP data.

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On the deflection coefficient value of plane plates

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Summary

In the plane rectangular plates loaded in compression within their median plane, their loss of stability occurs when the deflection unit stress σ_{cr} *has the magnitude given by the general expression:*

$$\sigma_{cr} = k\frac{\pi^2 D}{b^2 t}$$

where:

k - *deflection coefficient of the plate under investigation*:

$D = \frac{E t^3}{12(1-\mu^2)}$ - *bending stiffness of plate for a section of unit length and thickness* t;

b - *width of plate*.

The deflection coefficient depends on the following parameters:

- *manner of distributing compressive forces on the sides of the investigated plate;*
- *type of support on its contour;*
- *ratio of plate sides.*

Determining the deflection coefficient function of the three above mentioned parameters is rather complex, sometimes mathematically impossible, as it assumes the solving of a differential equation with partial derivatives of the 4th *order, with constant or variable coefficients. In many cases the energy method may be successfully applied, as its provides sufficiently accurate results, if more series terms are considered in the system of equations which establishes the critical values of compressive forces, where the plate deformation due to deflection is represented by a double trigonometric series.*

On the other hand, in current design work, the SR 1911-98 gives very simple relations for the calculation of deflection coefficients in deflection stability of plates check-up, which however, give results that sometimes are off from the value established by using the energy method. The paper compares the values obtained by using the two calculus methods.

1. INTRODUCTION

A plate deflected by certain forces acting on the median plane (Fig. 1) is represented by the equation (1):

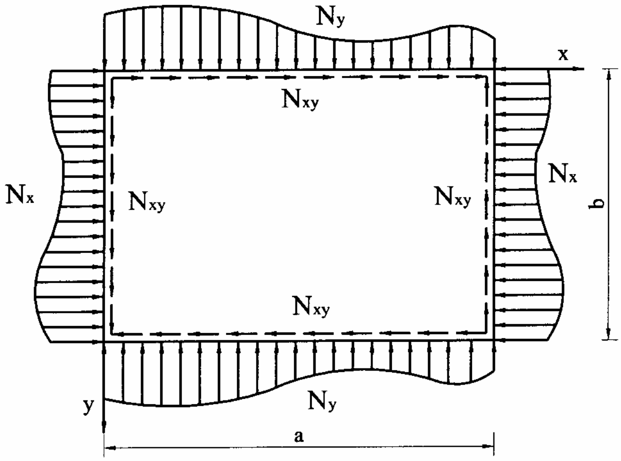

Fig. 1

$$\frac{\partial^4 w}{\partial x^4}+2\frac{\partial^4 w}{\partial x^2\partial y^2}+\frac{\partial^4 w}{\partial y^4}=\frac{1}{D}\left(N_x\frac{\partial^2 w}{\partial x^2}+2N_{xy}\frac{\partial^2 w}{\partial x\partial y}+N_y\frac{\partial^2 w}{\partial y^2}\right) \qquad (1)$$

w - displacement of plate, normal to its plane, due to deflection (bending deflection);

N_x, N_{xy}, N_y - forces in the median plane of plate, on the unit of side length, which can produce its deflection.

The values of the forces N_x, N_{xy}, N_y which cause the plate deflection – the critical values – result by solving equation (1) and depend on their distribution on the plate contour, the contour conditions and its dimensions. Solving this equation is quite difficult, especially so when the loads N_x, N_{xy}, N_y are variable along the plate sides.

2. DEFLECTION OF PLATES OVER TWO OPPOSITE SIDES

The case of the plate girders webs loaded over two opposite sides by a force N_x, given by relation (2) and distributed along the sides, as shown in Fig. 2, is an interesting one.

$$N_x = N_0\left(1-\beta\frac{y}{b}\right) \tag{2}$$

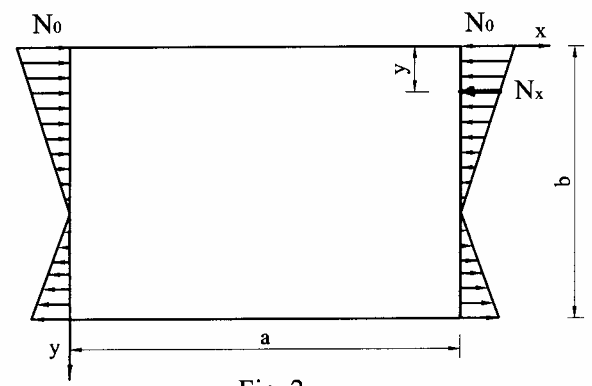

Fig. 2

The shape of N_x load distribution depends on the value of the coefficient β:
$\beta = 0$ - uniform compression;
$\beta = 2$ - pure bending;
$0 < \beta < 2$ - eccentric compression.

For this case of loading, equation (1) becomes:

$$\frac{\partial^4 w}{\partial x^4} + 2\cdot\frac{\partial^4 w}{\partial x^2\,\partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{1}{D}N_x\frac{\partial^2 w}{\partial x^2} \tag{3}$$

The solving of equation (3) with N_x, given by (2) is difficult, as the coefficient $\left(\frac{N_x}{D}\right)$ is variable (excepting case $\beta = 0$). In determining the critical value of the compressive force N_0 it can use the energy method, according to which when the plate is deflected, the rise in energy when the plate is being deflected ΔU, is equal to the work of the exterior forces acting on the plate ΔT, condition which results in $(N_0)_{cr}$. The expressions of the two variables for a plate loaded as shown in Fig. 1 are:

$$\Delta U = \frac{1}{2}D\int_0^a\int_0^b\left\{\left(\frac{\partial^2 w}{\partial x^2}+\frac{\partial^2 w}{\partial y^2}\right)^2 - 2(1-\mu)\left[\frac{\partial^2 w}{\partial x^2}\,\frac{\partial^2 w}{\partial y^2} - \left(\frac{\partial^2 w}{\partial x\,\partial y}\right)^2\right]\right\}dx\,dy \tag{4}$$

$$\Delta T = \frac{1}{2}\int_0^a\int_0^b \left[N_x \left(\frac{\partial^2 w}{\partial x^2} \right)^2 + N_y \left(\frac{\partial^2 w}{\partial y^2} \right)^2 + 2N_{xy} \frac{\partial w}{\partial x}\frac{\partial w}{\partial y} \right] dx\, dy \tag{5}$$

The relations (4) and (5), according to (2), $N_{xy} = N_y = 0$ are introduced for the plate in Fig. 2, while w, considering the plate articulated over its contour, can be represented by a double trigonometric series:

$$w = \sum_{m=1}^{\infty}\sum_{n=1}^{\infty} a_{mn} \sin\frac{m\pi x}{a} \sin\frac{n\pi y}{b} \tag{6}$$

$m, n = 1, 2, 3, \ldots$ have the significance of a number of deflecting semi-waves in direction x and y, respectively.

It can be seen that expression (6) satisfies the contour conditions, i.e., along the sides $x = 0, x = a, y = 0, y = b$, the deflection w and the bending moments M_x and M_y are equal to zero.

By equalizing relations (4) and (5) we obtain the expression of the critical deflection load:

$$(N_0)_{cr} = \frac{\pi^4 D \sum_{m=1}^{\infty}\sum_{n=1}^{\infty} a_{mn}^2 \left(\frac{m^2}{a^2} + \frac{n^2}{b^2} \right)^2}{\sum_{m=1}^{\infty}\sum_{n=1}^{\infty} a_{mn}^2 \frac{m^2\pi^2}{a^2} - \frac{\beta}{2}\sum_{m=1}^{\infty} \frac{m^2\pi^2}{a^2}\left[\sum_{n=1}^{\infty} a_{mn}^2 - \frac{32}{\pi^2}\sum_{n=1}^{\infty}\sum_{i}^{\infty} \frac{a_{mn}\cdot a_{mi}\cdot n\cdot i}{\left(n^2 - i^2\right)^2} \right]} \tag{7}$$

where for “i” are taken values that make $n \pm i$ an odd number.

The minimum value of $(N_0)_{cr}$ may be obtained from the condition:

$$\frac{\partial\left((N_0)_{cr}\right)}{\partial a_{mn}} = 0 \tag{8}$$

we obtain a linear equations system of the form:

$$D a_{mn} \pi^4 \left(\frac{m^2}{a^2} + \frac{n^2}{b^2} \right)^2 = \sigma_{cr}\, t \left\{ a_{mn} \frac{m^2\pi^2}{a^2} - \frac{\beta}{2} \frac{m^2\pi^2}{a^2} \left[a_{mn} - \frac{16}{\pi^2} \sum_{i}^{\infty} \frac{a_{mi}\cdot n\cdot i}{\left(n^2 - i^2\right)^2} \right] \right\} \tag{9}$$

where

$$\sigma_{cr}\, t = (N_0)_{cr} \tag{10}$$

The unknowns in system (9) are the coefficients a_{mn} of the trigonometric series (6) and if it is considered that the plate is deflected ($w \neq 0$), then $a_{mn} \neq 0$, which assumes that the determinant of system (9) is being annulled. The value of σ_{cr} results from this condition.

If $m = 1$ in (9) (deflection of plate along its axis is taken as a deflection semi-wave), yields the system:

$$a_{1n}\left[\left(1+n^2\frac{a^2}{b^2}\right)^2-\sigma_{cr}\frac{a^2 t}{\pi^2 D}\left(1-\frac{\beta}{2}\right)\right]-8\beta\sigma_{cr}\frac{a^2 t}{\pi^2 D}\sum_i^{\infty}\frac{a_{li}\cdot n\cdot i}{\left(n^2-i^2\right)^2}=0 \qquad (11)$$

The more equations are considered in system (11), the more accurate the value of σ_{cr}. It has been noticed that quite accurate values for σ_{cr} will be found when $n = 1,2,3$ (third approximation) is taken in (11), the fourth approximation giving values which do not differ significantly from the third approximation.
As in the literature the expression:

$$\sigma_{cr} = k\frac{\pi^2 D}{b^2 t} \qquad (12)$$

is commonly used for σ_{cr}, by introducing (12) in (11) and making, in turn, $n = 1$ $(i = 2)$, $n = 2$, $(i = 1,3)$ and $n = 3$ $(i = 2)$ we can obtain the system:

$$\begin{cases} a_{11}\left[\left(1+\frac{a^2}{b^2}\right)^2-\left(1-\frac{\beta}{2}\right)\frac{a^2}{b^2}k\right]-a_{12}\frac{16\beta}{9\pi^2}\frac{a^2}{b^2}k=0 \\ -a_{11}\frac{16\beta}{9\pi^2}\frac{a^2}{b^2}k+a_{12}\left[\left(1+4\frac{a^2}{b^2}\right)^2-\left(1-\frac{\beta}{2}\right)\frac{a^2}{b^2}k\right]-a_{13}\frac{48\beta}{25\pi^2}\frac{a^2}{b^2}k=0 \\ -a_{12}\frac{48\beta}{25\pi^2}\frac{a^2}{b^2}k+a_{13}\left[\left(1+9\frac{a^2}{b^2}\right)-\left(1-\frac{\beta}{2}\right)\frac{a^2}{b^2}k\right]=0 \end{cases} \qquad (13)$$

Giving various values for β (see relation (2)) and various a/b ratios, we obtain, by equalizing the system (13) determinant to zero, the deflection coefficient k values shown in Table 1.

Observation. For long plates $\left(\frac{a}{b} > 1\right)$, the k coefficient values determined for $m = 1$ are higher than those calculated for $m = 2$, that is, the long plates get deflected by two deflecting semi-waves. The values of the k coefficient for $m = 2$ can be determined by writing a system of linear equations (9) where $m = 2$ is introduced and in which, for $n = 1, 2, 3$ a system similar to system (13) will be particularized. The values of k coefficient have been calculated for the ratios $a/b = 1;\ 1.5;\ 2$ (Table 1).

Table 1

β \ a/b		0.4	0.6	0.8	1.0	1.5	2.0
2	m = 1	29.3635	24.178	24.627	27.132	39.559	58.368
	m = 2	-	-	-	25.634	24.142	50.964
4/3	m = 1	18.889	12.977	11.241	11.013	13.317	17.913
	m = 2	-	-	-	15.047	11.467	11.013
1	m = 1	15.157	9.744	8.132	7.812	9.249	12.35
	m = 2	-	-	-	11.627	8.368	7.812
1/3	m = 1	10.041	6.151	5.115	4.793	5.63	7.497
	m = 2	-	-	-	7.475	5.205	4.795
0	m = 1	8.41	5.138	4.202	4.0	4.694	6.25
	m = 2	-	-	-	6.25	4.340	4.0

3. VALUES OF DEFLECTION COEFFICIENT ACCORDING TO SR 1911 – 98

For the cases of plate loading and the ratio of the investigated panel shown in Table 1, SR 1911 – 98 gives simplified calculus relations with which we may obtain the values of the deflection coefficient from Table 2.

Table 2

β \ a/b	0.4	0.6	0.8	1.0	1.5	2.0
2	28.933	24.160	23.9	23.9	23.9	23.9
4/3	18.125	12.370	11.093	10.835	10.835	10.835
1	16.055	9.808	8.023	7.636	7.636	7.636
1/3	10.00	6.107	4.995	4.755	4.755	4.755
0	8.41	5.139	4.202	4.00	4.00	4.00

By comparing the values of the deflection coefficients in the two tables we can see that they are quite close in short plates $\left(\frac{a}{b} < 1\right)$ and types of loading that near uniform compression $\beta = 0$. Larger differences may result in ratios $\frac{a}{b} \geq 1$ and types of loads that near pure bending $(\beta = 2)$, for instance, for the ratio $\frac{a}{b} = 2$ and $\beta = 2$ in Table 1, $k = 50.964$, while in Table 2 $k = 23.9$, a more restrictive value if compared to the accurate value.

4. CONCLUSIONS

The relations used in calculating the deflection coefficient in rectangular plates loaded along two opposite sides as presented in SR 1911 – 98 give sufficiently accurate values for short plates and less accurate ones for long plates, which are within accepted limits in the sense that they give the critical values of the unit stresses in plate loading that are lower than the real ones.

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Performance in pipe culverts design

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Summary

The paper proposes solutions for a rapid and economical design of pipe culverts, so to ensure an optimum flow at the maximum hydraulic capacity. The detailed analysis of various types of water flow through pipe culverts and of main factors is realised. Then, the study is directed to the box pipe culverts with semi-forced flow. In order to design the culvert size, computer programs have been created.

KEYWORDS: culvert hydraulic category, semi-forced box culvert, backwater curve, hydraulic jump, culvert size, critical depth, design nomographs.

1. INTRODUCTION

Culverts represent 65% of the total length of the crossing realised through bridges and culverts. As a consequence, it results a high cost of the infrastructure. For this reason, it is necessary to design the culverts according to economic feasibility criteria both in the constructive solution analysis and in the calculation of the culvert outlet.

If a culvert cannot convey all of the incoming water, then the water will flow over or around the pipe, or simply back up behind the culvert creating a pond or reservoir. If any of these conditions were unacceptable, then the proper culvert dimensions and number of culverts must be selected prior to installation in order to convey all of the anticipated water through the pipe(s). This calculation helps the designer size culverts as well as presents an inlet water depth versus discharge rating curve.

Inlet control means that flow through the culvert is limited by culvert entrance characteristics. Outlet control means that flow through the culvert is limited by friction between the water current and the culvert barrel. The term "outlet control" is a bit of a misnomer because friction along the entire length of the culvert is as important as the tailwater depth. Inlet control most often occurs for short, smooth, or greatly downward sloping culverts. Outlet control governs for long, rough, or slightly sloping culverts. The type of control also depends on the discharge: inlet control may govern for a certain range of flows, while outlet control may govern for other discharges. If the discharge is large enough, water could go over the road.

A powerful design ensures the embankment stability and the protection against flooding. Pipe culverts represent a simple and more economical way of crossing watercourses, suitable for low water discharges.

2. CLASSES OF CULVERTS

The hydraulic design and the constructive solution depend on the type of culverts.
According to the height of the road embankment h_E, culverts can be divided into **twin slab culverts** (or surface culverts), if $h_E = 0$ and **pipe culverts**, if $h_E \geq 0.50$ m (Fig. 1).
According to the cross section form, pipe culverts can be classified into rectangular (box) (Fig. 2), circular, ovoid, other forms.

In order to use the whole capacity of the cross section, to diminish the hydraulic resistance at the culvert entrance and to protect the embankments from water erosion, parallel works meant to guide the flow towards the culverts are necessary. These works implicitly modify the velocity coefficient (φ) and the contraction coefficient (ε).

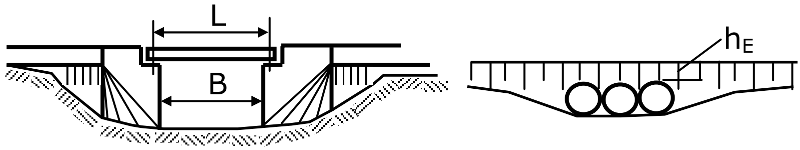
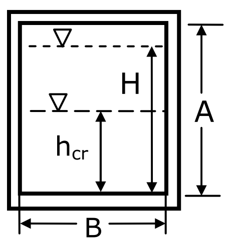


Figure 1. Twin slab culverts and pipe culverts

According to the headwater depth, pipe culverts fall into several categories: **free flow pipe culverts**, if $H < (1.2\ldots1.4)A$ and **pressure flow pipe culverts**, if $H > 1.2A$. Their water races transition system depends on the various hydraulic, geometrical and constructive parameters characterising the construction. The minimum value of A must be 1 m.

Figure 2. Cross-section of box culvert



The flow types and the different kinds of level water races transition at **free flow pipe culvert** are:

1) **Non-submerged free flow**, when $h_t < (1.2\ldots1.4)\,h_{cr} + i_{cr}\,(L - l_c)$. It is characterised by two forms of water races transition, realised through:

a) backwater curve, if $h_t \le h_{cr}$; in this case, the design depth will be considered equal with the critic depth: $h_{calc} = h_{cr}$ (Fig. 3);

b) free hydraulic jump, if $h_t > h_{cr}$, the design depth is tailwater depth $h_{calc} = h_t$.

2) **Submerged free flow**, when $h_t > (1.2\ldots1.4)\,h_{cr} + i_{cr}\,(L - l_c)$. In this case, the transition is realised through a submerged hydraulic jump and the design depth is $h_{calc} = h_t$.

From hydraulic point of view, free pipe culverts function like **broad sill weirs** with lateral contraction, whose sill height is considered to be null. The hydraulic design of the free pipe culverts relies on the hypothesis that the culvert slope is equal with the critical slope $\left(i_0 = i_{cr}\right)$.

The classification of the **pressure flow culverts** is, according to the inlet coefficient, the following: **forced culverts**, when $h_{calc} = A$ and **semi-forced culverts**, when $h_{calc} < A$.

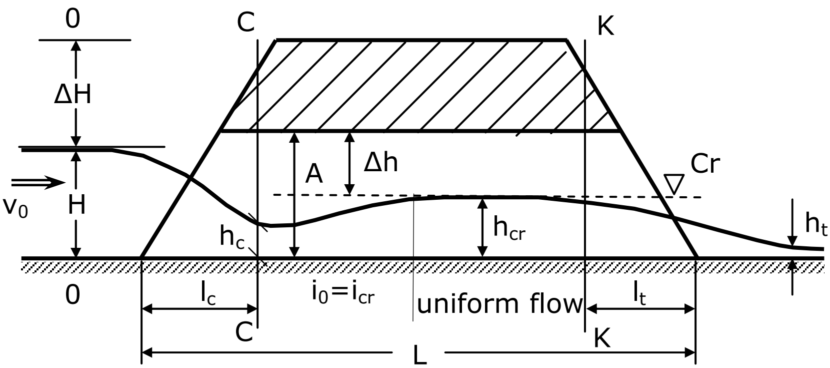


Figure 3. Free culvert flow: non-submerged flow with backwater curve
Symbols: $l_c = (1.5\ldots2.5)\,(H - h_c)$ = the *vena contracta* position; h_{cr} = the critical depth, $l_t = (2\ldots2.5)\,(h_{cr} - h_t)$ = the outlet zone length; h_c = the *vena contracta* depth; $l_{ef} = L - l_c - l_t$ = the effective zone length; h_t = tailwater (downstream water depth).

The downstream water depth influences the water's flowing through **forced culverts** and engenders two forms of outlets: **non-submerged outlet**, if $h_t < A$ and **submerged outlet**, if $h_t > A$.

Forced culverts are characterised by hydraulic conditions analogous to those of the **nozzles**; but they differ from nozzles by the fact that the head loss is taken into account.

The types of flow and water races transition forms of the **semi-forced culverts** depend on the flow state (Fig. 4), namely:

1) As far as the **quiet state** is concerned (when $i_0 < i_{cr}$), there are three forms of water surface:
a) backwater curve (when $h_t < h_{cr}$), for which $h_{calc} = h_{cr}$;
b) free hydraulic jump (when $h_t < h_c^{''}$), for which $h_{calc} = h_{cr}$;
c) submerged hydraulic jump (when $h_t > h_c^{''}$), for which $h_{calc} = h_t$.

2) The **rapid state** (when $i_0 > i_{cr}$); the open level is a downstream decreasing curve. In this case, $h_{calc} = h_0$, where h_0 represents the depth of the normal water level corresponding to i_0.

Semi-forced pipe culverts work similarly to **large orifices**.

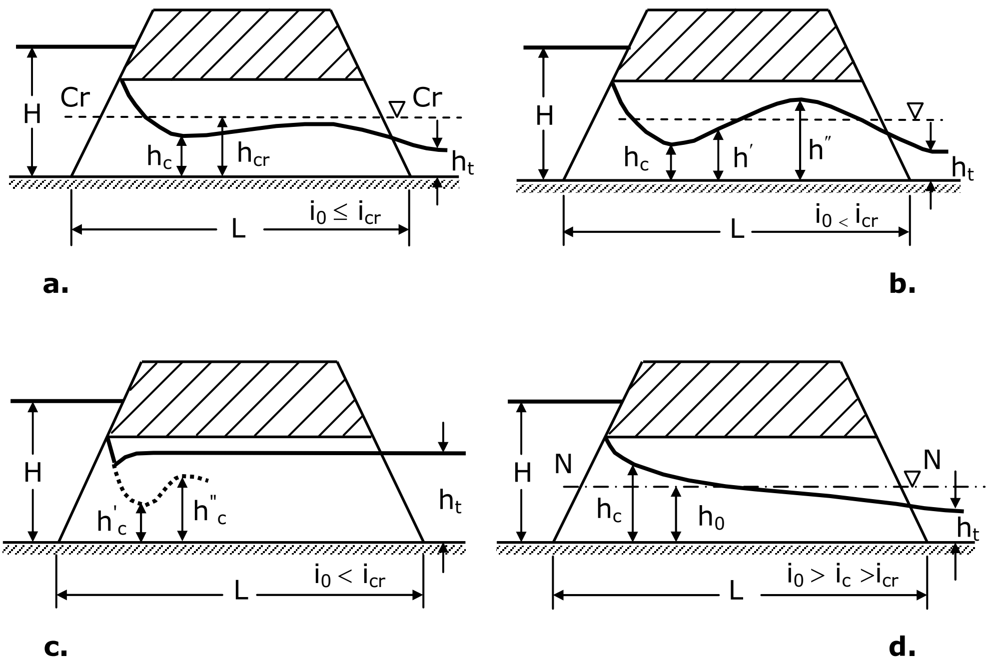


Figure 4. Semi-forced culvert flow
a.- backwater curve; b.- free hydraulic jump; c.- submerged hydraulic jump; d.- rapid state
$h^{'}$, $h^{''}$ = the conjugates depths.

3. SEMI-FORCED RECTANGULAR CULVERT

3.1. Concept of a Reliable Hydraulic Design

The hydraulic design of the pipe culverts is made according to the hypothesis assuming the existence of a prismatic riverbed ($\partial\omega/\partial L = 0$).

The initial specific data concerning the permissible velocity, the inlet coefficient and the dimension ratio and the vertical contraction are necessary. The slope and pipe roughness are major factors in determining whether the flow.

The stream occupies the whole pipe section only at the inlet; as for the rest of the construction, the section is partially full, the stream occupies just 50…60% of the cross section. Consequently, the usual methods used for twin slab culverts cannot be applied any more.

Three basic steps are required to design a culvert: determine required flow, select pipe size, and calculate flow velocity.

a) The **flow** through culverts is a result of storm water runoff. Runoff in small watersheds depends on the drainage area, the rainfall intensity, and the coefficient of runoff. Rainfall intensity is a function of storm frequency and duration. Typically, a storm frequency of 50 years is used for culvert design. The coefficient of runoff represents the ratio of runoff to rainfall and considers ground cover, soil type, and topography. The required flow used for design in a culvert is the maximum flow resulting from the collection of runoff at any point in the system.

b) The **permissible velocity** is chosen according to the bed and the concrete characteristics. The water velocity is determined at a point in the culvert, where the flow depth (h_{calc}) is equal to the critical depth or tailwater depth, whichever is smaller.
Outlet velocity of a culvert for the cases a,b,d in Fig. 4 is higher than the velocity of the natural streambed because of improved hydraulic conditions. Excessive energy from the higher velocity may damage or erode the streambed that could result in loss of foundation support for the culvert structure. The energy may be absorbed by tailwater if proper conditions exist, by streambed protection, or dissipated with precast flared end sections and velocity reduction rings.

c) The **pipe size selection** depends on the culvert location and the optimal flow conditions. These conditions determine the ratio between the pipe height and its width:

$$m = \frac{A}{B} = 0.5\ldots1.5 \qquad (1)$$

The hydraulic design is based on Bernoulli's relation and on the calculation principles of large orifices.

d) The **critic slope**:

$$i_{cr} = \frac{g\,P_{cr}}{B\,C_{cr}^2} \tag{2}$$

where the Chézy coefficient C is calculated from Manning’ relation:

$$C = \frac{1}{n} R_h^{\frac{1}{6}} \tag{3}$$

e) The **critic depth**:

$$h_{cr} = \sqrt[3]{\frac{Q^2}{g\,B^2}} = 0{,}467\,\sqrt[3]{\frac{Q^2}{B^2}} \tag{4}$$

f) The **contracted section** results from the Bernoulli's relation applied between the inlet section and the contracted section. At the entrance, the water has the depth H, but at contracted section the water depth is h_c. The Coriolis coefficient may be considered α = 1. It results:

$$h_c = H + \frac{\alpha\, v_0^2}{2\,g} - \frac{h_{cr}^3}{2\,\varepsilon^2\,\varphi^2\,h_c^2} \tag{5}$$

The contracted section is located in the vicinity of the inlet and his depths may be approximated as:

$$h_c = \varepsilon_V\; A \tag{6}$$

where ε_V represents the vertical contraction coefficient (Table 1).

Table 1. Velocity coefficient (φ) and contraction coefficient ($\varepsilon, \varepsilon_V$) of pipe culverts

Entrance type	φ	Lateral contraction ε		Vertical contraction ε_V
		Free culvert	Semi-forced culvert	
Simple tympanum	0.80	0.85	0.60	0.60
Cone quarters or wings	0.85	0.90	0.64	0.64
Hydraulic tympanum	0.95	1.00	0.65	0.65

g) The **conjugated depths** result from the hydraulic jump equation:

$$(h')^2\,h'' + h'\,(h'')^2 = \frac{2\,Q^2}{g\,B^2} \tag{7}$$

Finally, the mathematical formulae for velocity and water discharge are get.

h) The **water velocity**:

$$v = \varphi \sqrt{2g(H - h_c)} = \varphi \sqrt{2g(H - \varepsilon_V \ A)} \tag{8}$$

where φ represents the velocity coefficient (Table 1).

i) The **culvert letting out capacity**:

$$Q = \varepsilon_c \ \omega v = \varepsilon_c \ \omega \varphi \sqrt{2g(H - \varepsilon_V \ A)} = \mu A B \sqrt{2g(H - \varepsilon_V \ A)} \tag{9}$$

where μ represents the discharge coefficient.

j) The **water depth** (*H*) at the culvert input can be derived from the Equation (4):

$$H = \frac{Q^2}{2g\mu^2 A^2 B^2} + \varepsilon_V \ A \tag{10}$$

3.2. Analytical results

In order to design semi-forced box culverts, a program based on the Equations (8), (9), (10) has been conceived. The graphics were plotted for the same purpose. For example, the design nomographs if ε_V = 0.6 are presented on the Figure 5.

In order to use easily a large range of water discharges, these nomographs have been plotted so: for low values of flow in decimal scale (Figure 5), and for large values in a logarithmic scale (Figure 6).

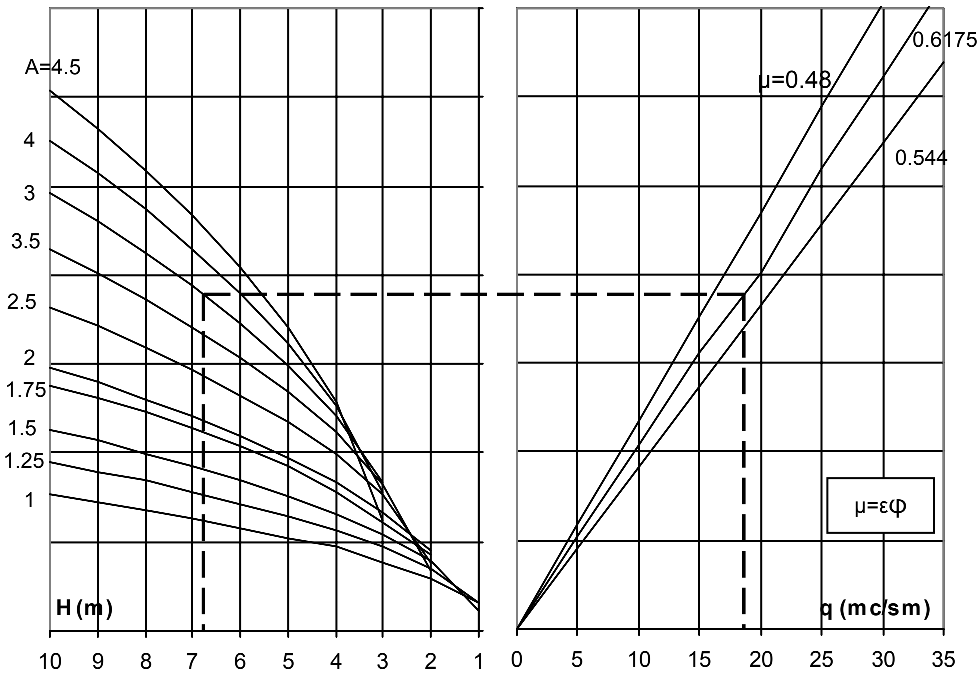


Figure 5. Nomographs for design of semiforced box culvert (decimal scale)

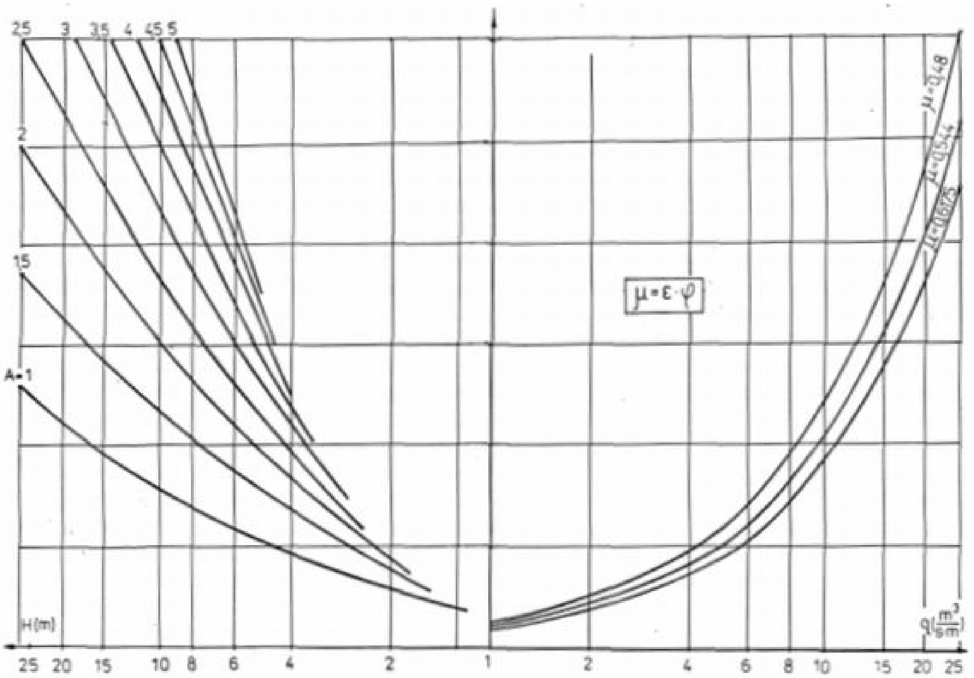


Figure 6. Nomographs for design of semiforced box culvert (logarithmic scale)

4. CONCLUSIONS

A reliable design of box culverts is necessary, first of all, to define their hydraulic category. In this way, we may be able to accurately appreciate the initial data and the calculation steps. In order to obtain a greater degree of precision and accuracy and to enlarge the value domain of the examined parameters, a computer program able to offer the best design solution has been created. Also, the corresponding nomographs were plotted as a possibility of calculation. Both, the computer program and the graphics correspond to the construction solutions, which ensure the optimum flow at a maximum culvert capacity. These nomographs are reliable and efficient, enabling a fast design.

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Quantitative evaluation -forecasting of the technical condition of flexible-semi-rigid road pavements.

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Summary

The paper presents the study of the evaluation indexes for flexible/semi-rigid road pavements , based on the statistical laws, for two RO-LTPP road sectors , located on the National Road network. The correlation coefficients and the statistical residues involved, are finally permitting the authors to emphasize the opportunity of the use of this methodology , in order to assure an objective programming of the maintenance works, in case of flexible/semi-rigid road pavements .

1. INTRODUCTION

1.1. The pavement layer distress condition for flexible/semi-rigid road pavements is expressed in terms of the following indexes:

* IEST- index of structural condition, determined based on the evaluation of the structural distresses , which are affecting the pavement , as follows: (1) distress caused by the fatigue of the pavement structure, (2) alligator cracking,(3) longitudinal cracking, (4) patching, (5)ruting,(6)pumping phenomena and (7)potholes.

* IESU- index of surface condition, determined based on the evaluation of surface distresses, as follows: (8) edge cracking,(9)cracks transmitted in the area of working joints,(10)transverse cracking,(11)potholes affecting the surface layer, (12) corrugations (13) bleeding, (14) polished surface , (15) raveling , (16) drop of shoulders.

*IG- index of global evaluation

1.2. The evaluation of the distress condition is performed on homogenous sectors.

1.2.1. In addition to the provisions of the specific technical norms/1/ besides the traffic, the type of pavement structure and the last year of intervention(surface treatment, reinforcing, rehabilitation) the following criteria are justified to be considered at the selection of homogenous sectors: the hydrological conditions, the type of soil in subgrade, the geometric features of the road.

1.2.2. It is recommended that the division of the road in homogenous sectors to be done in accordance with the AASHTO methodology.

1.3. The homogenous sectors are further divided in homogenous sub-sectors, based on the real condition of the pavement, expressed in qualitative terms of “good”, “mediocre” and “bad”/1/.

1.4. For every type of distress the following severity degrees are assigned: low

severity (r), medium severity (M), high severity (R)/1/.

1.5 The frequency of the apparition of the distress (p) is classified as.ing. follows: occasionally (O),moderate (M) or frequent (F)

2. CALCULATION OF THE EVALUATION INDEXES

2.1.The IEST index is calculated with the relation :

$$IEST = 100-\sum P*D \qquad (1)$$

where:

P*D- “the deducted points” calculated function of the structural distress type (1…7), the degree of severity (r, M, R) and the frequency of their apparition (O, M, F)/1/.

2.2. The IESU index is calculated with the relation:

$$IEST = 100-\sum P*D \qquad (2)$$

IESU = “the deducted points” calculated function of the type of surface distress (8…16), the degree of severity and the frequency of their apparition /1/

2.3 The IG index is obtained with the relation :

$$IG = ((IEST)*(IESU))^{0.5} \qquad (3)$$

3. CHARACTERIZATION OF THE ROAD PAVEMENT CONDITION

The characterization of road pavement condition, by is presented in Table 1 , /1/.

Table 1

Qualificative	IEST	IESU	IG
Good	>90 ;≤100	>90	>90 ;<95
Mediocre	>80 ;≤90	>75 ;≤90	>77 ;≤90
Bad	≤80	≤75	≤77

4. CASE STUDY

4.1. The investigations for the determination of the evaluation indexes has been made on two national road sectors, having the characteristics presented in Table 2.

Table 2

NR	Type of pavement structure	Year of constructii on	Maintenance works-TSB*	Climatic Type	Sector's position	Period of investi-gations
18	Flexible	1977	1995;1997	III	189+000-189+150	1998…2004
24	Semi-rigid	1979	1992 ;1996	I	96+687-96+837	1997…2004

. Asphalt surface treatment

4.2. The investigations of the road pavement condition has been performed for the summer/spring and fall seasons.

Table 3

Season	DN 18				DN 24			
	years	IEST	IESU	IG	years	IEST	IESU	IG
summer	0.9	77.5	79.1	78.1	0.8	99.5	77.8	88.0
	1.8	73.4	82.6	77.6	2.9	93.9	66.9	79.0
	2.9	66.4	72.2	68.9	3.9	90.3	63.8	75.8
	3.9	69.3	77.1	72.5	4.9	91.4	63.9	76.3
	4.8	67.4	73.5	70.4	5.8	85.5	61.3	72.1
	5.7	64.5	74.5	68.7	6.8	91.9	76.3	83.7
	5.9	65.6	70.3	66.9	7.0	86.7	68.9	77.2
	6.8	72.2	76.3	74.0	7.9	88.7	73.3	80.5
Spring/ fall	2.6	72.0	74.0	72.6	1.6	97.7	66.6	83.9
	3.6	68.8	80.4	74.0	3.6	91.9	69.6	79.9
	5.0	63.3	70.4	65.7	4.6	91.2	66.5	77.6
	6.5	72.3	77.3	74.6	6.0	85.4	60.8	71.5
	-	-	-	-	7.6	87.8	76.2	81.7

4.3. In Table 3 the values of the evaluation indexes are presented, with the specification of the period, in years, from the date of the achievement of the TBS til the date of the investigations. The number of years since the construction of the pavement layer is derived from Table 2.

4.4. The statistical distributions for the evaluation indexes, for the whole monitoring period are presented in Table 4.

Table 4

Statistical Distribution	DN 18 / DN 24		
	IEST	IESU	IG
Weibull	3.14/0.848;6.44/4.22; - 6.44	-	-
	2.74/0.882;3.73/3.80; -0.29	6.67/0.575;11.64/8.83; -7.54	-
P(4)	2.36/0.938;2.66/3.99; -0.46	4.16/0.724;4.01/6.18; -0.82	4.00/0.875;6.04/4.79; + 0.13
	3.12/0.885;3.78/3.32; +0.11	4.45/0.881;7.41/5.61; +0.36	3.92/0.857;5.75/4.70; +0.28
P(3)	2.59/0.900;3.89/3.78: -1.71	3.61/0.723;4.32/6.25; -0.67	3.47/0.875;6.15/3.51; +0.07
	2.73/0.883;4.10/3.43; +0.40	4.30/0.849;9.33/6.27; +2.15	3.62/0.835;6.90/5.09; +1.35
P(2)	2.41/0.890;3.98/3.58; -2.72	3.42/0.681;5.55/4.84; -2.26	3.73/0.813;6.42/6.64; -2.96
	2.46/0.882;3.93/3.65; +0.11	3.89/0.846;9.83/5.63; +3.00	3.25/0.834;7.11/4.82; +1.71
Other distributions	(5)2.25/0.925;2.78/4.74; -0.11	(6)2.73/0.852;4.07/5.18; -2.54	(7)4.39/0.661;8.60/9.51; -8.59
	(8)2.36/0.869;3.74/4.31; -0.90	(10)4.0/0.836;10.12/5.98; +2.67	(9)3.49/0.805;9.22/5.44; -0.20

For each index/statistical distribution type, data from Table 4 , pursuing the existing order have the following significance: S- the standard error, R- coefficient of correlation, r_+ -the maximum statistical residue %, r_- - the maximum statistical residue %, for the final year, between y calculated/measured (y being the evaluation index).

4.5. Evaluation of the confidence level for the forecast at the stage “n”, based on the statistical distribution at the stage “n-1” is presented in Table 5 (for the national road DN18) and in Table 6 (for the National Road DN24): the season in which the investigation have been performed, the investigation stage “n-1”, the type of statistical distribution (pt.4.6), S, R, and Δ (%)- the difference between the calculated/ measured values of the index, recoded during the investigation stage “n”.

Table 5

Season	X(years)/ No.of data	IEST	IESU	IG
Summer	0,9...3,9 (4)	(6)0,00/1,000;-0,64	(6)0,00/1,000;+4,90	(6)0,00/1,000;+17,00
		(3)2,51/0,954;+7,83	(3)6,39/0,538;+3,37	(3)4,33/0,821;+17,69
	0,9 4,8 (5)	(14)2,10/0,921;+4,23	(6)2,90/0,938;+1,29	(5)3,24/0,968;-13,06
		(2)3,07/0,942;+5,11	(2)0,00/1,000;+7,07	(4)5,98/0,889;-12,90
		(3)2,23/0,939;+9,62	(3)4,56/0,644;-2,15	(3)4,16/0,892;-15,01
	0,9...5,7 (6)	(8)1,91/0,936;-0,67	(6)2,12/0,939;+6,73	(7)3,89/0,813;-1,10
		(2)2,23/0,956;-3,02	(4)3,96/0,766;+5,38	(4)4,97/0,851;+0,15
		(3)2,41/0,923;+0,20	(1)5,37/0,787;+2,54	(2)4,82/0,860;+3,14
		(1)2,60/0,971;-5,70	(3)3,75/0,667;+5,58	(3)4,29/0,832;-0,29
	0,9...5,9 (7)	(4)2,11/0,948;-8,31	(12)3,15/0,738;-7,09	(2)4,00/0,869;+0,65
		(3)2,09/0,932;-8,22	(2)4,06/0,738;-6,72	(1)4,87/0,870;+7,55
		(8)1,71/0,943;-10,65	(3)3,52/0,738;-6,82	(3)3,71/0,847;-8,93
Spring/ fall	2,6...5,0 (3)	(5)0,00/1,000; 0,00	(5)0,00/1,000; 0,00	(3)5,40/0,649;-1,26
		(3)2,82/0,921;-0,68	(4)0,00/1,000;-2,46	(4)0,00/1,000;-4,72
		(4)0,00/1,000;-5,99	-	(6)0,00/1,000;-5,11

Table 6

Season	X(years)/ No.of data	IEST	IESU	IG
Summer	0,9…4,9 (4)	(13)0,00/1,000;+2,95	(13)0,00/1,000;-3,86	(13)0,00/1,000;-0,33
		(3) 1,54/0,976;+6,86	(3) 0,54/0998;+5,98	(3) 0,87/0,996;+6,88
	0,9…5,8 (5)	(11)1,72/0,957;-8,44	(8) 0,77/0,994;-20,28	(7) 1,28/0,982;-13,71
		(4) 3,00/0,956;-8,13	(4) 1,33/0,994;-20,40	(4) 2,12/0,984;-14,50
		(3) 2,10/0,957;-8,71	(3) 1,18/0,991;-17,98	(3) 1,64/0,981;-13,74
	0,9…7,0 (7)	(7) 2,54/0,867;-1,39	(3) 3,99/0,860;+7,42	(4) 4,96/0,754;-4,09
		(4) 3,16/0,877;-0,46	(4) 6,89/0,647;-8,80	(3) 3,47/0,848;+4,24
		(3) 2,75/0,876;+0,26	(1) 5,32/0,877;+15,44	(2) 3,83/0,861;+9,54
		(2) 3,11/0,881;+2,86	(2) 4,34/0,877;+15,21	(1)4,69//0,862;+11,84
Spring/ fall	1,6…6,0 (4)	(4) 0,00/1,000;-7,96	(4) 0,00/1,000;-31,14	(4) 0,00/1,000;-21,54
		(3) 1,52/0,984;-8,89	(3) 0,79/0,992;-36,85	(15)0,58/0,997;-21,70
		(15)1,53/0,984;-8,57	(15)0,70//0,993;-34,45	(3) 0,53/0,998;-22,59

4.6.Statistical distributions mentioned (in tables are as follows: (1) P(4)- polynom of the forth degree, (2)-P(3), (3)-P(2), (4) Weibull, (5) rational distribution, (6) MMF, (7) logarithmic distribution, (8) inverse logarithmic distribution, (9) Hoerl distribution,(10) inverse P(2) distribution, (12) inverse liniar distribution, (13) Richard distribution, (14) exponential distribution, (15) Gauss distribution. These statistical distributions are presented in detail in Table 7.

Table 7

Crt. No..	Distribution No	Statistical distribution	Formulas	Cathegory
1	(5)	Rational	$Y=(a+b*x)/(1+c*x+d*x^2)$	
2	(6)	MMF	$Y=(a*b+c*x^d)/(b+x^d)$	Sigma type
3	(9)	Hoerl	$Y=a*(b^x)(x^c)$	power
4	(13)	Richard	$Y=a/(1+\exp(b-c*x))^{(1/d)}$	Sigma type
5	(14)	Exponential	$Y=a*b^{(1/x)}$	

5. CONCLUSIONS

5.1. For the investigated road sectors, the evolution of the evaluation indexes (IEST, IESU, IG) follows with a satisfactory confidence level, the appropriate patterns of various statistical distributions.

5.2. A minimum of four stages of investigations is necessary in order to assure the necessary confidence level of the statistical distribution involved.

5.3. The type of the statistical correlation can be modified according the stage of the investigation but the confidence level has to be assured for all the investigation stages involved.

5.4. The forecast of the technical condition of the road pavement, by using statistical distributions and the creation in this way, of the possibility of achieving an objective programming/prioritization of maintenance works, is justified to be used in place of the actual practice which in accordance with the specific technical instructions, involve only the admittance of the maintenance intervals .

5.5. The forecasting of the IEST, IESU and IG values is useful/necessary on the representative sectors (summer/spring-fall) because the IEST influence during spring-fall seasons is significant.

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Analize parametrice privind amplificarea seismică a împingerii active asupra zidurilor de sprijin

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Abstract

Parametric Analyses of Seismic Amplification of Active Pressure on Retaining Walls

The paper presents the results of a numerical analysis of the main parameters influence on the soil active seismic pressure. The parameters taken into consideration were those that depend on: a) the nature of the soil - by the internal friction coefficient; b) structure's geometry - by the skew of that parameter against the vertical of the upstream parameter, location configuration given by the skew angle of the back – of-the-wall against the horizontal; c) the interaction of soil and the face of the retaining wall - given by the angle of friction of the two.

At the same time, the maximum skew angle of the supported soil whose value is equal to the difference between the internal friction angle φ and the apparent rotation angle of the gravitational axis during the seismic event, $\theta = \arctan k_h$, was taken into account. From among the results that were obtained, this paper tackles the values of the seismic amplification coefficient, defined by the ratio of the active seismic pressure and the static one. This ratio is important from a practical point of view in evaluating the weight of the retaining wall in the seismic analysis, on which depends the amplification coefficient given by the weight of the wall, determined by the static pressure in order to obtain the necessary weight of the retaining wall during a seismic event.

The amplification coefficient that corresponds to the ultimate value of the skew angle of the back-of-wall soil oscillates around the unit, which is justified by the compensatory effect of the internal friction angle φ and the apparent rotation θ of the gravitational axis.

1. INTRODUCERE

Zidurile de sprijin prin funcţiunile pe care le îndeplinesc sunt construcţii de protecţie şi asigurare; proiectarea şi execuția lor implică responsabilitate şi în acest sens aprofundarea privind comportarea acestor lucrări, îndeosebi la cutremure, este necesară şi de utilitate aplicativă.

Răspunsul seismic al zidului de sprijin este caracterizat îndeosebi de împingerea pământului, care este acțiunea fundamentală asupra acestor lucrări şi de stabilitatea şi de rezistența sistemului construcţie de sprijinire-teren. În continuare ne referim la prima problemă, împingerea seismică a pământului. Studii teoretice în acest sens sunt numeroase şi de asemenea şi semnificative rezultate experimentale, dar metode dinamice specifice care să poată fi utilizate în proiectare nu au fost puse încă la punct. O metodă cvasistatică pentru proiectarea zidurilor de sprijin la acțiunea seismică, care se aplică tot mai frecvent în prezent, este metoda Mononobe-Okabe, care în timp a fost perfecţionată, dezvoltată şi susținută de cercetări în domeniu. Nu de mult timp a început să fie aplicată şi în țara noastră de către unii specialişti la proiectarea zidurilor de sprijin cu funcţiuni importante. În procesele decizionale ale proiectării se întâmpină unele dificultăți datorită îndeosebi necunoaşterii în suficientă măsură a influenței parametrilor care determină valorile împingerii seismice a pământului, prin variațiile lor şi prin modul de direcţionare. De altfel, în acest sens există încă lacune şi privind măsura influenței parametrilor asupra împingerii statice a pământurilor. În continuare se prezintă unele rezultate privind studiul parametric al împingerii active seismice a pământului în cazul zidurilor de sprijin de greutate care se folosesc cu o mare frecvență. Rezultatele reținute se referă cu precădere la amplificarea împingerii active a pământului generată de cutremure.

2. METODA CVASISTSTICĂ PENTRU CALCULUL ÎMPINGERII ACTIVE SEISMICE A PĂMÂNTULUI

Pentru bazele teoretice ale problemei se pot consulta unele dintre referințele bibliografice. În continuare se fac unele precizări necesare scopului urmărit. Inițiatorii metodei mențin ipotezele formulate de Coulomb pentru problema statică enunțate în toate lucrările de specialitate, de exemplu [1], [2], [3], [9]. În plus, se propune să se ia în considerare acțiunea seismului printr-o densitate volumică de forță $\gamma k s$ urmare a efectelor inerțiale generate de cutremur. Se admite că în starea limită, la o deplasare minimală a zidului, se formează o suprafață de cedare - lunecare în masivul de pământ sprijinit. Ca şi în cazul static, s-a admis că această suprafață este un plan care pleacă din colțul de jos al zidului şi este înclinat cu unghiul α față de orizontală. Elementele geometrice ale zidului sunt date în fig. 1. Forțele care acționează volumul de pământ care lunecă şi împingerea care se transmite zidului de sprijin sunt arătate în fig. 2. Se menționează că în problemele de stabilitate intervin greutatea proprie a zidului şi forța de inerție generată de cutremur asupra acestuia.

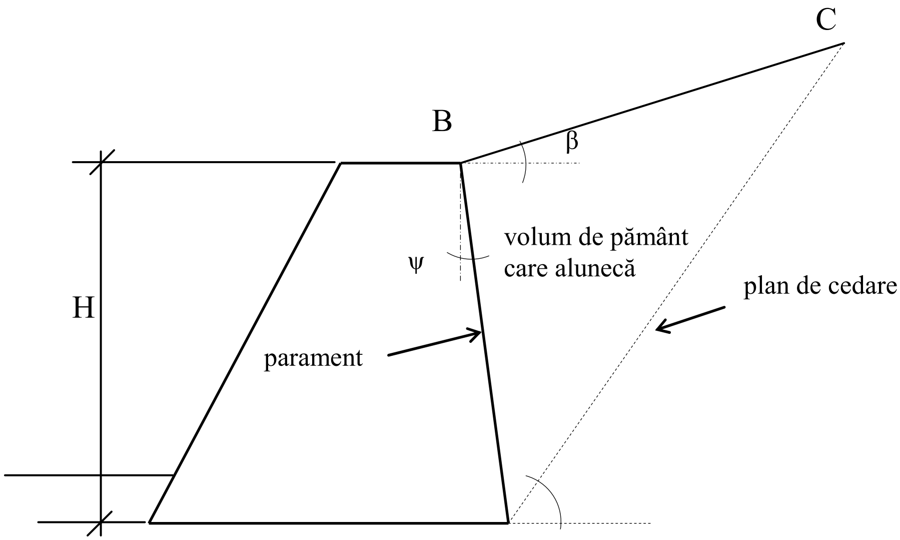

Fig. 1. Elementele geometrice ale zidului

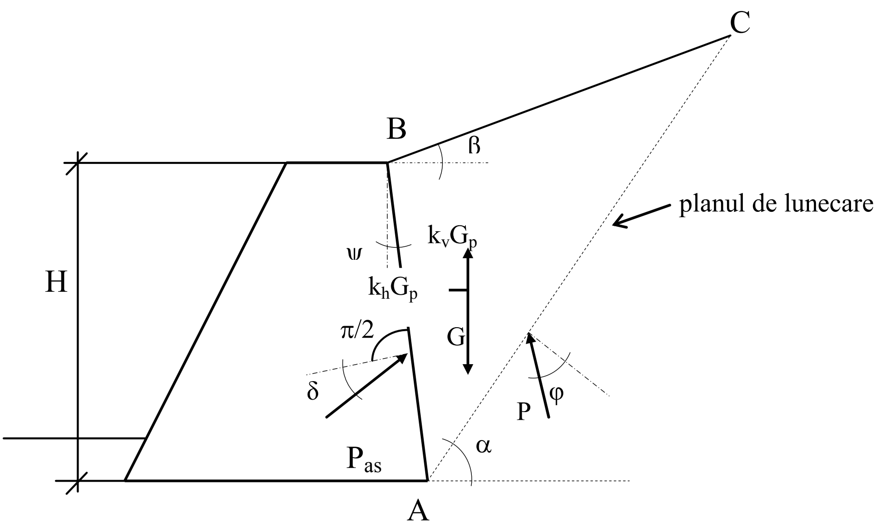

Fig. 2. Forțele care acționează volumul de pământ

Împingerea activă seismică se determină cu relația

$$P_{as} = \frac{1}{2} \cdot \gamma \cdot H^2 \cdot K_{as} \qquad (1)$$

unde:

H - înălțimea zidului;

γ - greutatea specifică a pământului;

K_{as} - coeficientul seismic al împingerii active şi care are expresia:

$$K_{as} = \frac{\cos^2(\varphi - \theta - \psi)}{\cos\theta \cdot \cos^2\psi \cdot \cos(\delta + \psi + \theta) \cdot \left[1 + \sqrt{\frac{\sin(\varphi + \delta) \cdot \sin(\varphi - \beta - \theta)}{\cos(\delta + \psi + \theta) \cdot \cos(\beta - \psi)}}\right]^2} \qquad (2)$$

în care:

φ - unghiul de frecare internă a pământului;

ψ - înclinarea paramentului de contact cu pământul față de verticală;

β - unghiul de înclinare al terenului din spatele zidului față de planul orizontal;

δ - unghiul dintre normala la paramentul zidului de sprijin şi rezultanta P_{as}, cu semnificația de coeficient de frecare dintre pământ şi zidul de sprijin;

θ - unghiul aparent de rotire al axei gravitaționale (Mononobe)

$$\theta = \text{arctg}\frac{k_h}{1 - k_v} \qquad (3)$$

k_h – coeficientul seismic al accelerației orizontale;

k_v - coeficientul seismic al accelerației verticale.

Pentru proiectare este indicat să se determine θ fără influența accelerației verticale, deci:

$$\theta = \arctan k_h \qquad (4)$$

Influența şi interacțiunile sunt multiple şi nu pot fi evidențiate prin formule analitice. Pentru evaluări efective sunt necesare analize numerice ordonate şi direcționate. Există câteva studii [4], [12], care pun în evidență faptul că efectul δ este redus şi poate fi ignorat, iar pentru perete vertical sunt date variații ale coeficientului împingerii seismice funcție de k_h şi φ.

Variații pe scara seismică din România au fost determinate în referatul [14]. Analizele parametrice referitor la împingerea seismică la zidurile de sprijin au fost dezvoltate şi amplificate sistematic de autorul acestui articol, iar dintre acestea se prezintă o secvenţă referitor la factorul de amplificare seismică.

3. ANALIZE PARAMETRICE EFECTUATE

S-au considerat 3 amplasamente seismice semnificative determinate prin k_h= 0,07; 0,12; 0,20. Pentru fiecare amplasament s-au stabilit variaţii ale parametrilor φ, δ, β, ψ. S-au considerat valorile cel mai frecvent întâlnite în practică. Pentru φ s-au luat valori între 15° ÷ 35° cu pasul 5°. Valorile uzuale pentru β au fost 0°, 5°, 10°, 15°, 20°, iar pentru ψ 0°, 10°, 15° şi -10°. Unghiul de frecare δ s-a raportat la φ şi s-au considerat valorile 0, δ=φ/2, δ=2φ/3. S-au adoptat şi indicaţiile din normele japoneze şi anume, conform cărora în timpul cutremurelor, unghiul δ se diminuează faţă de cazul static şi diminuarea depinde de intensitatea cutremurului; astfel s-au adoptat de exemplu la δ = 2φ/3 - static, δ = 0,54, pentru k_h = 0,07, δ = 0,337φ, pentru k_h = 0,12 şi δ = 0,167φ pentru k_h = 0,20.

Rezultatele numerice au fost tabelate, cu care s-au putut realiza şi grafice de variaţie. În continuare se prezintă valori numerice privind variaţia coeficienţilor de amplificare seismică în raport cu limitele practice considerate ale parametrilor specificaţi.

Tabelul 1. Factorul de amplificare seismică f_{as} pentru k_h=0.07-0.12; δ=(1/2-1/3)φ; ψ=0°

	k_h = 0,07 δ=0,5φ ψ=0°					k_h=0,12 δ=0,333φ ψ=0°				
	φ=15°	φ=20°	φ = 25°	φ = 30°	φ = 35°	φ =15°	φ =20°	φ = 25°	φ = 30°	φ = 35°
β = 0°	1,092	1,11	1,122	1,124	1,143	1,208	1,219	1,239	1,245	1,262
β = 5°	1,132	1,131	1,1367	1,14	1,004	1,106	1,255	1,256	1,273	1,271
β = 10°	1,236	1,16	1,148	1,147	0,927		1,342	1,3	1,285	1,33
β = 15°		1,283	1	0,995	0,991			1,182	1,141	1,137
β = 20°			1,337	1,201	1,189				1,454	1,375
β = max	1,039	1,012	1,023	1,034	0,966	0,982	0,997	1,017	1,024	0,961

Tabelul 2. Factorul de amplificare seismică f_{as} pentru k_h=0.20-0.07; δ=(1/6-1/2)φ; ψ=0°-10°

	k_h = 0,20 δ=0,167φ ψ=0°					k_h=0,07 δ=0,5φ ψ=10°				
	φ=15°	φ=20°	φ = 25°	φ = 30°	φ = 35°	φ =15°	φ =20°	φ = 25°	φ = 30°	φ = 35°
β = 0°	1,435	1,418	1,442	1,45	1,578	1,085	1,092	1,079	1,104	1,1
β = 5°		1,549	1,499	1,511	1,512	1,181	1,107	1,105	1,104	1,107
β = 10°			1,655	1,574	1,564	1,212	1,132	1,087	1,119	1,116
β = 15°				1,498	1,42		1,265	1,162	1,135	1,125
β = 20°					1,848			1,293	1,27	1,148
β = max	1,001	0,968	1,065	1,095	1,044	0,980	0,99	0,994	0,991	1,001

Tabelul 3. Factorul de amplificare seismică f_{as} pentru k_h=0.12-0.20; δ=(1/3-1/6)φ; ψ=10°

	k_h = 0,12 δ=0,333φ ψ=10°					k_h=0,20 δ=0,167φ ψ=10°				
	φ=15°	φ=20°	φ = 25°	φ = 30°	φ = 35°	φ =15°	φ =20°	φ = 25°	φ = 30°	φ = 35°
β = 0°	1,168	1,179	1,25	1,261	1,26	1,362	1,333	1,33	1,343	1,35
β = 5°	0,809	1,204	1,2	1,2	1,2		1,455	1,394	1,381	1,383
β = 10°		1,287	1,237	1,22	1,208			1,535	1,443	1,419
β = 15°			1,339	1,265	1,24				1,608	1,5
β = 20°				1,376	1,286					1,682
β = max	0,945	0,958	0,957	0,962	0,98		0,961	0,985	1,01	1,031

Tabelul 4. Factorul de amplificare seismică f_{as} pentru k_h=0.07-0.12; δ=(1/2-1/3)φ; ψ=15°

	k_h = 0,07 δ=0,5φ ψ=15°					k_h=0,12 δ=0,333φ ψ=15°				
	φ=15°	φ=20°	φ = 25°	φ = 30°	φ = 35°	φ =15°	φ =20°	φ = 25°	φ = 30°	φ = 35°
β = 0°	1,076	1,084	1,084	1,087	1,081	1,146	1,151	1,152	1,153	1,154
β = 5°	1,102	1,096	1,098	1,094	1,085	1,218	1,185	1,176	1,171	1,613
β = 10°	1,207	1,126	1,103	1,103	0,998		1,265	1,21	1,396	1,072
β = 15°		1,25	1,282	1,122	1,109			1,31	1,234	1,206
β = 20°			1,288	1,161	1,132				1,34	1,25
β = max	0,970	0,977	0,98	1,018	0,983	0,926	0,931	0,934	0,95	0,95

Tabel 5. Factorul de amplificare seismică f_{as} pentru k_h=0.20-0.07; δ=(1/6-1/2)φ; ψ=15°,-10°

	k_h = 0,20 δ=0,167φ ψ=15°					k_h=0,07 δ=0,5φ ψ= -10°				
	φ=15°	φ=20°	φ = 25°	φ = 30°	φ = 35°	φ =15°	φ =20°	φ = 25°	φ = 30°	φ = 35°
β = 0°	1,331	1,3	1,296	1,294	1,293	1,141	1,142	1,156	1,186	1,183
β = 5°		1,415	1,351	1,33	1,319	1,163	1,168	1,175	1,192	1,184
β = 10°			1,486	1,398	1,238	1,272	1,196	1,459	1,191	1,194
β = 15°				1,551	1,433		1,33	1,706	1,215	1,225
β = 20°					1,612			1,382	1,27	1,25
β = max			0,925	0,948	0,969	1,000	1	1,001	0,936	0,917

Tabelul 6. Factorul de amplificare seismică f_{as} pentru k_h=0.12-0.20; δ=(1/3-1/6)φ; ψ=-10°

	k_h = 0,12 δ=0,333φ ψ= -10°					k_h=0,20 δ=0,167φ ψ= -10°				
	φ=15°	φ=20°	φ = 25°	φ = 30°	φ = 35°	φ =15°	φ =20°	φ = 25°	φ = 30°	φ = 35°
β = 0°	1,263	1,288	1,308	1,329	1,361	1,539	1,533	1,566	1,623	1,678
β = 5°	1,335	1,317	1,333	1,355	1,368		1,676	1,638	1,665	1,705
β = 10°		1,55	1,382	1,374	1,393			1,821	1,752	1,756
β = 15°			1,487	1,415	1,432				1,951	1,878
β = 20°				1,562	1,487					1,746
β = max	1,028	1,044	1,069	1,021	1,109	1,070	1,115	1,164	1,142	1,171

Determinările s-au efectuat grupate pe fiecare amplasament seismic dat de k_h, permiţând analize directe utile şi pentru proiectare. Datele obţinute permit şi studii comparative între amplasamentele seismice prin care se pune în evidenţă gradul de intensitate seismică al zonei.

Din analiza datelor obţinute se desprind o serie de concluzii, care se prezintă sintetic.

În timpul cutremurului împingerea activă pe paramentul zidului de sprijin creşte, iar această amplificare poate fi măsurată prin intermediul factorului de amplificare seismic f_{as}, care se defineşte ca raport între împingerea dinamică a pământului ce se manifestă în timpul cutremurului şi împingerea statică, deci

$$f_{as} = \frac{P_{as}}{P_a} = \frac{K_{as}}{K_a} \tag{5}$$

Se prezintă aceşti coeficienți pentru trei amplasamente semnificative la care corespund coeficienții seismici egali cu: 0,07; 0,12; 0,20. Din punct de vedere practic între aceste valori se pot folosi tehnici de interpolare, cu rezultate foarte bune, aşa cum a rezultat din analizele efectuate.

Din valorile tabelate se constată:

⇒ Coeficienții de amplificare cresc cu φ şi cu atât mai mult cu cât coeficientul seismic este mai mare; astfel la k_h = 0,20 se ajunge aproape la dublarea împingerii seismice active față de cazul static;

⇒ Coeficienții de amplificare cresc cu unghiul β de înclinare a suprafeței terenului din spatele zidului; creşterile pot ajunge la 20%;

⇒ Un efect favorabil asupra amplificării seismice îl are unghiul ψ de înclinare a paramentului de sprijinire dacă acesta este pozitiv; dacă înclinarea paramentului este către terenul sprijinit, factorul de amplificare capătă valori mai mari, comparabil cu paramentul vertical;

⇒ Unghiul $\beta_{min} = \varphi - \theta$, limitează foarte mult înclinarea terenului din spatele zidului şi aceasta se datorează unghiului de frecare internă, deoarece θ depinde de coeficientul seismic de zonă;

⇒ Pentru $\beta = \beta_{max}$ factorii de amplificare rămân aproape staționari oscilând în jurul unității; acest fapt se datorează esențial compensării dintre efectul unghiului de frecare internă φ şi cel al rotirii aparente θ a axei gravitaționale şi implicit al coeficientului seismic.

4. CONCLUZII

Prin studiile efectuate şi consecințele cele mai semnificative subliniate rezultă o serie de constatări utile în concepția şi dimensionarea zidurilor de sprijin.

⇒ Pentru lucrările de pe căile de comunicație auto, c.f. din zone accidentate sau montane este foarte important să se cunoască coeficientul de frecare internă şi îndeosebi β_{max}, întrucât prin depăşirea acestuia există riscul compromiterii lucrărilor de sprijinire.

⇒ Lucrările de sprijinire din zona podurilor unde intervin şi umpluturi, trebuie analizate cu atenție atât din punct de vedere al coeficienților de frecare internă existenți, respectiv realizați, cât şi din punctul de vedere al pantelor terenului sprijinit, cei doi factori şi coeficientul seismic formând un grup de parametri cu efecte conjugate.

⇒ Executarea unor lucrări de sprijinire cu pante către teren îndeosebi din pământ armat sunt expuse la amplificări seismice mari şi trebuie atent verificată asigurarea.

⇒ Alte probleme pot fi de asemenea asociate atât pentru proiectare cât şi în cazul execuției şi care nestăpânite pot conduce la avarii ale acestor lucrări.

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Information System for Time-Domain Monitoring of the Bridge - Riverbed Interaction Area Behavior

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Summary

This paper is an attempt to emphasize the importance of time domain studying of the bridge-riverbed interaction area, a place where several events and phenomena (which often had brought to a serious degradation or bridges collapse) happen.

In the introductory part, several damaged bridges are presented. The main cause of damages was the foundation erosion phenomena together with the thalweg descent. This last phenomenon is mainly due to wrong exploitation of gravel pit, which lead to the destruction of the bottom sills in different places.

The second part of the article analyzes the information system concept based on computer, together with the importance of databases. This implies the analysis of the information shown in the phenomena analysis produced in the bridge-riverbed interaction area.

The final part of the paper presents the necessary databases for an efficient activity of the information system, specific to the bridge-river interaction area. Databases containing information regarding the bridge behavior in time-domain in interaction with the crossed water course channel are highlighted.

KEYWORDS: bridge, information system, data base, channel, riverbed works, bottom sill.

1. INTRODUCTION

The technical events which took place in the last few years, at several road bridges, have emphasized a wide scale of damages. These damages have been analyzed during the last years, by several analysts in books, congresses and symposiums of bridges and roads in specialty journals.

Between the most important observed degradations the low water descent in the bridge area can be particularly mentioned. The consequence is of a visible stripping of the foundation – superstructure joint and even of the superior part of the foundation system. Other phenomena are: the thalweg deepening, that is associated with the low-water descent; the borders fracture; the change in plan of the water course lay-out due to the border fracture and the displacement of the minor riverbed toward the riverbank; the defense construction works sub-wash and water directing rigid construction works sub-wash etc.

These types of damages have been observed at an important number of bridges. For a better understanding, next a range of typical bridges in this situation are presented:

a. Bridge in Roman, National Road 2, 328 km +437m (fig.1, 2 and 3)

Figure 1. Pier no 6 of the bridge crossing Moldova River, from Roman, displaced due to an erosion process.

Figure 2. View of the bridge way from Roman, crossing Moldova River, after the pier no. 4 failure.

b. The bridge from Clit, Natl. Road 3E, 56 km +414m (Figs. 4 and 5)

Figure 4. Abutment displacement from the initial position with scour of the con’s quarter.

Figure 5. The earthwork creeping behind the abutment

c) Bridge from Sadova, on Natl. Road 17A, 0km+412m (fig.6)

Figure 6. The low-water descent in the bridge area

d) The bridge on Natl. Road 17A, 77 km +803m, at Dorneşti;

Figure 7. The Dorneşti Bridge: the change in plan of the water course lay-out, before and after rehabilitation

e) The bridge over Siret River from Ion Creangă on County Road 207C, 6km+075m (fig.8)

Figure 8. The bridge from Ion Creangă over Siret River – rehabilitation works and securitization of the pile no. 10, after the scour phenomena manifestation

2. INFORMATION SYSTEM – CONCEPT

The physical processes which take place in time in the bridge-riverbed interaction area at the level of the bridge elements and also in the riverbed impose their control and direction.

Also, it is known that these processes are being characterized by factual entities expressed in the form of numerical values or in the form of observations made by people or machines generally called data. In order to reduce the uncertainty and to understand several situations or phenomena, data must be transformed into information.

The information, as it is well known, represents the essential support of the leadership. It is necessary for establishing the strategy of the objectives and for examining their practice applicability.

In order to realize the presented needs for the study of the bridge-riverbed interaction area, creation of an informational system is necessary. For the goal to maintain the bridge during maintenance at the projected parameters, this system should contain entire activity complex through which the information is recorded, processed used. Because in this information system computers dominate, the system becomes an informatics system.

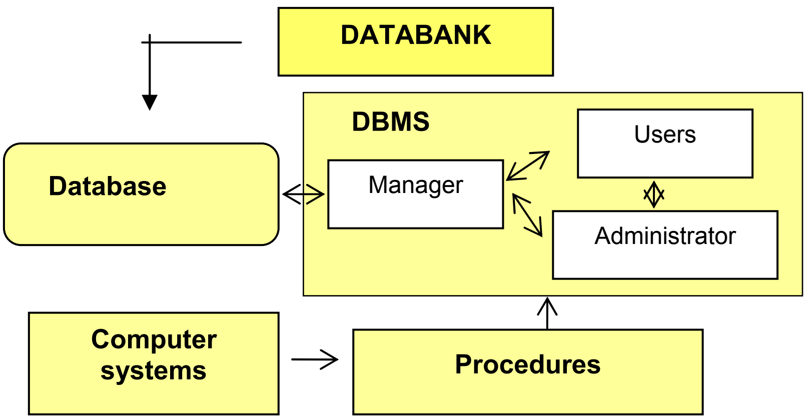


Figure 9. Information system

The databank plays an extremely important role in the information system. It is containing: the database, the system manager, the computer systems and the necessary procedures for the data administration.

At its turn, the system that manages the database (Database Management System or DBMS) is formed by three main components: the system users, the database administrator and the database manager.

2. CREATION OF THE INFORMATION SYSTEM

For a well functioning of the information system more databases are necessary, such as:

- The general database with information withdrawn from the construction project containing: the bridge name, year of construction, location and details regarding the site geomorphology, the bridge type from the construction point of view, the construction material, the construction elements, the geometrical elements of the bridge, the bridge image, fig. 10;

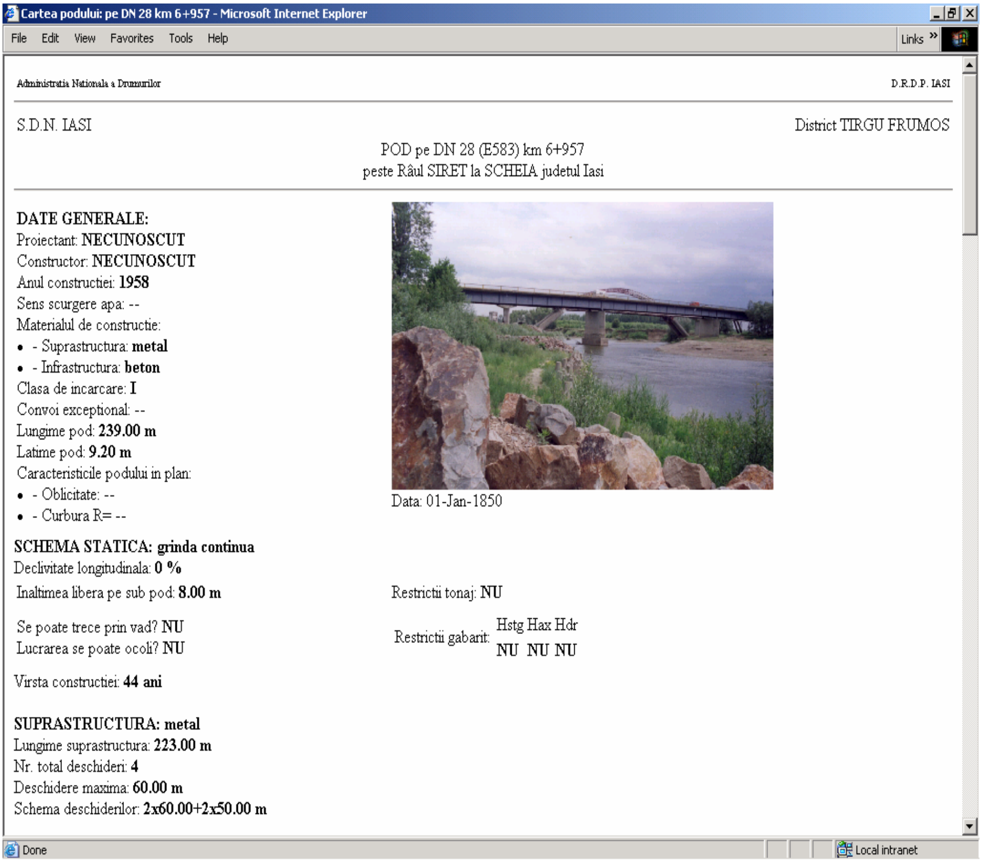
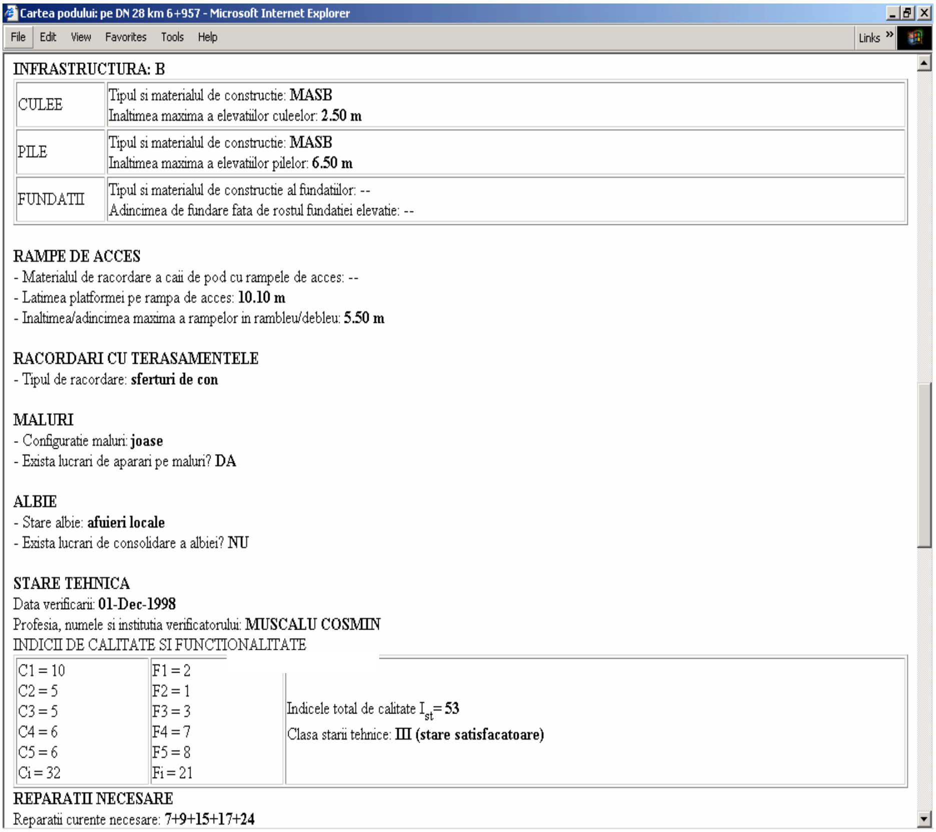


Figure 10. The database interface

- The topographic database containing the topographic location plan and of the neighborhood area, the inventory of the points measured on site, the resulted situation plan;
- The geographic database with the location details, the water hydrologic behavior and with geotechnical structure;
- The technical data which is identified from the bridge's technical book. The bridge's book must contain: identification data, fig.11.; geometric and

constructive elements regarding the infrastructure (abutment, pier, bearing block) and the superstructure; the bridge’s road; improvements in the bridge’s area; important data from the bridge’s history (design, construction, repairs and/or rehabilitation); time-domain monitoring of behavior according to the monitoring schedule, quantitative information regarding the behavior in time, analysis, the impairment determination and technical examination;

- The legislative database regarding the design, construction and management activity (maintenance and pursuit of the behavior in time);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INFRASTRUCTURA: B

CULEE	Tipul si materialul de constructie: **MASB** Inaltimea maxima a elevatiilor culeelor: **2.50 m**
PILE	Tipul si materialul de constructie: **MASB** Inaltimea maxima a elevatiilor pilelor: **6.50 m**
FUNDATII	Tipul si materialul de constructie al fundatiilor: -- Adincimea de fundare fata de rostul fundatiei elevatie: --

RAMPE DE ACCES
- Materialul de racordare a caii de pod cu rampele de acces: --
- Latimea platformei pe rampa de acces: **10.10 m**
- Inaltimea/adincimea maxima a rampelor in rambleu/debleu: **5.50 m**

RACORDARI CU TERASAMENTELE
- Tipul de racordare: **sferturi de con**

MALURI
- Configuratie maluri: **joase**
- Exista lucrari de aparari pe maluri? DA

ALBIE
- Stare albie: **afuieri locale**
- Exista lucrari de consolidare a albiei? NU

STARE TEHNICA
Data verificarii: **01-Dec-1998**
Profesia, numele si institutia verificatorului: MUSCALU COSMIN
INDICII DE CALITATE SI FUNCTIONALITATE

C1 = 10	F1 = 2	
C2 = 5	F2 = 1	
C3 = 5	F3 = 3	Indicele total de calitate I_{st} = **53**
C4 = 6	F4 = 7	Clasa starii tehnice: **III (stare satisfacatoare)**
C5 = 6	F5 = 8	
Ci = 32	Fi = 21	

REPARATII NECESARE
Reparatii curente necesare: 7+9+15+17+24

Figure 11. Database interface

- The data base regarding the bridge – riverbed interaction. Contains the data coming from the process of the in time determination of this area behavior: transversal profile in the bridge area, in uphill and downstream; the bottom sill state, the erosion state; the border defense work situation, of the jetty; the infrastructure’s in time displacement state; the monitoring of the thalweg evolution during the year: in spring, summer and autumn, fig. 12.

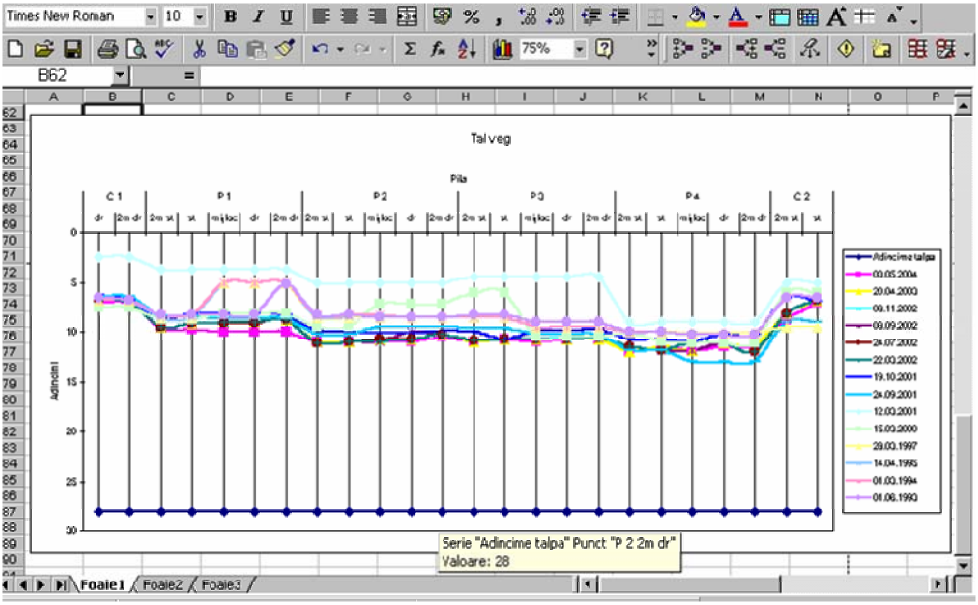

Figure 12. The thalweg situation reported to the foundation foot mark

3. CONCLUSIONS

1. Phenomena from the bridge – riverbed interaction area are very important, and examples of bridges seriously damaged from this cause are more and more numerous in our country.

2. Determination of the in time bridge-riverbed interaction area behavior leads to creation of an information system, where the data base have a very important role, in order to determine the right technical state of the bridges, so that the decisions stick to the reality.

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Iaşi, România, December 17, 2004

Road Bridges Performances Improvement through Bridge Engineering Processes Optimization

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Summary

In this paper a conception regarding the improvement of the road bridges performances through optimization of Bridge Engineering processes is presented.

In the first chapter of this paper the bridge network from our country considering the exploitation period is shown. It results that most of the bridges have a life period between 30 and 50 years.

In the second chapter a very rich bibliography connected with the paper theme is analyzed. It contains papers published at the following scientific meetings: Bridge and Roads Rehabilitation, 1999 and 2000, Cluj-Napoca; Timisoara Academic Days, 1999; The XI-th National Congress of Bridges and Roads in Romania, 2002, Timişoara; International Symposium Computational Civil Engineering 2003, Iasi; Bridge Maintenance and Consolidation, International Symposium, Bucureşti, 2004. A special place is dedicated to the presented doctoral thesis and to the published books during the last few years in our country.

In the third chapter the concept of total bridge realization is revealed. This concept is introduced in accordance with the System Theory and defined as a sequential process which includes multiple economical, technological and environmental aspects. Also, the Systems Engineering creates the grounds of scientific methods application in the terms of total definition of a bridge construction, regarding the design, construction and maintenance.

Based on this concept, the optimization technique finds it's place in all realization phases of a bridge, on several levels: in the case of a bridge design - the effective design and the rehabilitation design or bridge strengthening; in the operational phase - as a bridge system management subsystem etc.

A special place in this theory is devoted to the data information flow regarding the acquisition, processing / transformation, storage (in the bridge technical book and in the specific database) and utilization in the Bridge Engineering processes (by bridge management system). The paper ends with conclusions and the bibliography.

KEYWORDS: bridge, bridge engineering, design, in time determination of the bridge behavior, maintenance.

1. INTRODUCTION

In our country, approximately 65% from the total road traffic and 90% from international traffic is running on the national road network. In this case, it is necessary to mention the fact that during the last 10 years the total roads traffic volume in Romania has augmented 47%, and the statistics indicate a possible augmentation of 40% more in the near future. In this context, the experts in the area of the roads infrastructure consider the roads and bridges infrastructure development from two points of view: quantitative and qualitative.

The existence of an important number of bridges in our country: 3228 bridges on national roads, and approximately 30000 on communal and county roads raise issues related with their maintenance at parameters imposed by performance request of structural and functional nature, see Figure 1.

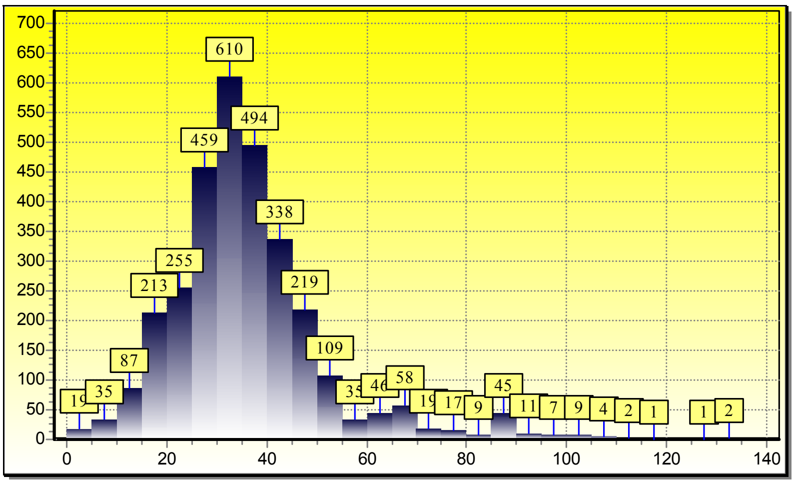

Figure 1. National bridge distribution on age groups

The requirement of structural safety refers to: avoiding a construction element or the entire structure’s collapse (i.e. requirements regarding the resistance, stability and ductility); and those regarding the functionality in maintenance (referring to avoiding excessive deformations and cracks openings).

Bridges are constructions extremely important for users. This is why, in case when one or more of their operational performances are outdated, discontinuities appear in the maintenance process. The repercussions in assurance of the communication way where they are implemented determine traffic disturbance. In the case of some damages, considering the fact that the bridges are also very expensive, their rehabilitation to the functionality state, they impose important financial efforts from the society.

2. THE RESEARCH STAGE IN THE BRIDGE ENGINEERING AREA

The strategy of the roads network administration contains, between other objectives, two of which are essential to determine the qualitative level. More exactly: knowing the technical state of the bridge network requires the exploitation parameters evaluation through non-destructive measurements with specific equipments and the technical and technological request evaluation. All of these impose the maintenance, at a certain level, of the exploitation parameters or the reconstruction, by adequate works, of the modified parameters by road traffic action or the climatic phenomena.

In our country, similar to the most developed countries in the world, the determination of the bridges' technical state and their reconstruction by repairing, consolidations or rehabilitation represents a constant preoccupation of the researchers in this area, designers, and engineers. The preoccupations previously mentioned are determined by technical-scientific initiatives (such as papers published for national and international scientific meetings), presented and recognized doctoral dissertation, books publication and norms.

During national scientific meetings, with or without international participation, the problem of bridges technical performances improvement was approached considering the following aspects:

- Strengthening solutions of the existing bridge superstructures, determination of bridge's behavior during maintenance, reinforced concrete bridge rehabilitation, determination by design of the life time. All of these objectives were presented at the symposium named *Bridges and Roads Rehabilitation, 1999, in Cluj-Napoca, Romania;*
- Stress state analysis in the concrete bridges plates, bridge design information, rehabilitation solutions by external prestress, methods for determination of bridges life period, aspects regarding the bridges with mixed structure reliability. All subjects have been considered at the *Timisoara Academic Days Symposium, 1999;*
- The bridge solutions optimization, consolidation and rehabilitation of road bridges, concrete bridges reliability and maintainability, systematic approach of the data base role in bridge administration. These solutions were analyzed at the *Bridges and Roads Rehabilitation Symposium*, *2000, Cluj-Napoca;*

- Static and dynamic tests of bridges, safety exploitation of road bridges, technical state prediction using statistical analysis, ways of obtaining data and data storage necessary to the bridge examination, rehabilitation and projection solutions of the bridges, contributions presented at the XI^{th} *National Congress for Roads and Bridges from Romania, 2002, Timişoara;*
- The bridge technical state, statistical modeling of the road bridges way, information flow in bridge engineering, concrete bridges degradation in time, approaches of the researchers at the *International Conference Constructions 2003, Cluj-Napoca*;
- The bridge disturbance process, using composite materials for the bridge rehabilitation, the technical bridge examination, the bridge administration system, studies published at the *International Symposium Computational Civil Engineering 2003, Iasi*;
- New materials and technologies used for bridge repairing and consolidation represented the theme of the *International Bridge Symposium, Bucureşti, 2004.*

It can be observed that, on internationally scale, a similar approach regarding the bridge exploitation period extension, the monitoring in time of bridge behavior, bridge maintenance, the use of new technologies and materials in the rehabilitation process is proposed. Therefore the authors have treated thsese topics at *Symposium on Metropolitan Habitats and Infrastructures*, in September 2004, at Shangai, China, a meeting organized by the International Association for Bridge and Structural Engineering.

At Novi Sad, in Serbia and Montenegro, International Association Bridges across the Danube has organized the *V^{th} International Bridge Conference*, June 2004, where a large area of problems regarding the design, construction, reconstruction and maintenance of the bridges (exactly the problems presented by the bridge engineering processes) have been analyzed.

In the Republic of Moldova, at Chişinau, from two in two years, a recognized international scientific meeting takes place. It is dealing with the actual problems of the urbanism and territorial fitting out. Regarding the bridge area, problems of the bridge design, seismic analysis of the bridges, bridge inspection and automatic adapting of the data resulted from this activity, the bridges examination etc,, were treated.

During the last years in Romania, many doctoral dissertations were presented. They were referred to the next subjects: *Bridge behavior under the dynamic action of the convoys* (Nicolae Malcoci, Iaşi, 2000); *The road bridges plates determination considering the other elements of the concrete bridge superstructure* (Corina Chiotan, Bucureşti, 2000); *Determination of technical state of the bridges by probabilistic methods* (Rodion Scînteie, Iaşi, 2001); *Evaluation of the impact of* *the art works on environment* (Ioan Gradinaru, Iaşi, 2001); *Composition and determination of the road bridges* (Cluj-Napoca, 2002); *Safety in bridge exploitation and rehabilitation* (Edward Petzek, Timişoara, 2000) etc.

The bridges behavior from the Romania roads network determined the change and the improvement of an important number of technical instructions and norms. It could be mentioned the ones referring to the public roads maintenance and renovation, AND 554-2002, the hydraulic design of the bridges and footbridges, PD 95-2002, technical instructions for the determination of technical state of a bridge, AND 522-2002 etc.

The studied bibliography for the present paper extended also to the books published during the last few years and which have a connection with the analyzed theme, among which we mention: Scînteie R., *The Work of Art Reliability*, Academic Society "Matei - Teiu Botez" Publishing House, Iasi, 2003; Scînteie R., *Database and algorithms for communication ways*, Academic Society "Matei - Teiu Botez" Publishing House, 2003; Mohora, C., and others. *The Production Simulation Systems*, Academy Publishing House, Bucureşti, 2001; Preoțescu, D., *Studies and reliability models with the use of economic indicators*, AGIR Publishing House, Bucureşti, 2001; Stănciulescu, Florin, *The Modeling of important complexity systems*, Technical Publishing House, Bucureşti, 2003 etc.

3. THE OPTIMIZATION OF PROCESSES FROM BRIDGE ENGINEERING

3.1. Total realization of the bridges

The choice of the final optimal solution for a bridge is obtained from multi-criteria analysis realized for several structures, each of them resulted from an optimization process.

A bridge approach, from technical point of view, cannot be issued by taking it out from the roads network context. This is why the methodology of total realization of a bridge concept, see Figure 2, introduced by the Systems Theory (of bridges) is defined by a sequential process which includes multiple aspects of economical, technological and environmental nature etc. Also the systems engineering proposes the application of scientific methods in terms of total definition of a bridge creation, regarding the design, execution and maintenance. So, naturally, the Systems Engineering is dealing with the fundamental problems of systems theory: analysis, synthesis and system leading.

A close analysis of the elements included in Figure 2, leads to the idea that the optimization is a concept which finds its place in all the phases of a bridge realization, on several levels: in the case of a bridge design (design level), as effective design, rehabilitation design or the bridges consolidation; in the case of operational phase, as subsystem of the bridge management system: project level and network level; in the selection process, by comparison, of the project alternatives (rehabilitation, for example) for a bridge and/or bridges projects from the network.

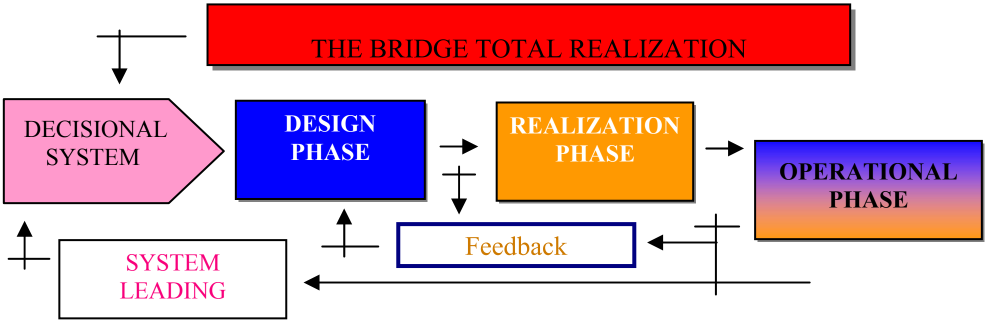

Figure 2. Bridge Engineering Cycles

By applying an optimization procedure at the bridges management system level (BMS) the strategies regarding the corrective maintenance activities that will be applied to the subsystems or to the bridge system can be determined. This could lead to the lowest cost on the exploitation period. The cost on the exploitation cycle considers the maintenance realized on the whole service time. Considering this approach, preventive maintenance actions can be started, before the appearance of major degradations in the bridge structure. On long term, their reparation might lead to much bigger costs.

The optimization procedure, specific to the exploitation procedure, must take into consideration the desired service level that a bridge must fulfill. Also, the service level will consider the volume and traffic structure which the bridge serves.

3.2. Information flow

The data acquisition process is determined in time-domain, considering the phase in which the bridge is considered, see Figure 3. This way, due to the process and procedures development from the operational state-maintenance system, by preventive and corrective procedures, by bridge revision operation, by the establishment of the technical state using quality indexes, by bridge expertise etc., a series of data regarding the bridge technical state with the help of quality indices are brought. After taking the decision regarding the bridge maintenance, by BMS, the maintenance project procedure, followed by the maintenance works is started. From these two phases data referring to the bridge will be brought. At the end of the maintenance works the works reception will take place that is another moment in when data will be taken. Finally, in the bridge historical evolution, after the maintenance phase, the procedures of system's determination in time of the bridge behavior follow.

Here there are some several examples and data that can be obtained:

- Data regarding the design, taken from consulting the following documents: the design theme, the bridge location and the notice which were the base of the project creation, technical characteristics of the bridge, execution details, designer, constructor or beneficiary's changes during the bridge execution; estimation breviaries (actions, estimation hypothesis, estimation results of dimensioning and checking); conditions of contract regarding the works execution.

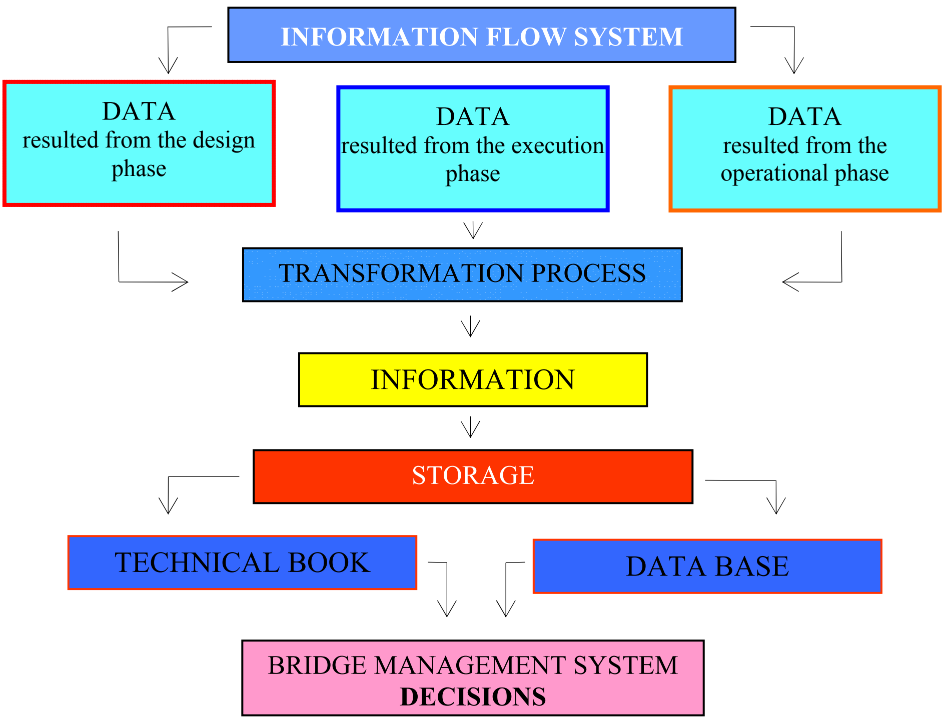

Figure 3. Components and interactions of information flow subsystem

- Data regarding the bridge execution that come from the next documents analysis: bridge execution license, protocol of location restoration, determinations regarding the used materials quality inspection, during the works; researches' results obtained after exceptional situations happened; journal of the main events: floods, earthquakes, exceptional temperatures etc.

- Data resulted from the study of the following documents referring to the works reception: reception protocol; documents realized due to the requests of the reception commission.
- Data obtained from technical analysis on the bridge behavior, instructions of the exploitation and maintenance in time domain: designers writing prevision regarding the bridge behavior following, exploitation and maintenance instructions; documents on which, after the final reception of the works, changes of the bridge towards the initial project were realized; project of the bridge special instrumentation data (static and/ or dynamic tests, time domain measurements), if exists; delivery – receiving protocol of the measurement installations between the performer and beneficiary, provided through the bridge special instrumentation project; the report with the annual and final conclusions on the special determinations; the events journal in which the next are included: the periodical checking results from the current determination; the checking results and measurements from the special determination; intervention measurements in the case of deficiency determination; exceptional events: earthquakes, floods etc.
- Data regarding the actual prescriptions referring to the bridge composition, considering the time for realization of execution project.
- Data regarding the materials' characteristics used for the bridge execution: for concrete – class, aggregates' granulometry, concrete type and quality, preparation method; for reinforcement – the brand and steel type, the steel's characteristics resulted from the provisional tests at construction site; for the steel from the metallic confections – brand and steel type, providers, the tests reports realized in laboratories, welding etc.
- Data regarding the geotechnical studies, hydrological studies, the area's seismicity.

3.3. Optimization of the design process

Through the design process an optimal system is wanted to be obtained. This desire is done in order to be the best possible system, corresponding to the existent development phase in a society at the time of a social order issue. The classical concepts of engineering design take into consideration only the technical criteria, which can lead to a bridge structure that must assure the resistance and stability in exploitation.

In the case of the reinforced concrete, the design procedure consists in: span number determination and their length; the piers' shape; inclination and their height; and also constructive principles on which the component elements dimensions are determined. Based on the actions the structure is analyzed. If the obtained results do not correspond to the imposed requirements, changes until obtaining determined limits takes place.

The modern concept for design will lead to an optimal solution only if it will consider the number of factors that influence it, more exactly the technical and economical criteria together with the esthetic conditions. The optimal design algorithm supposes a balanced combination of these criteria. In these conditions, strictly mathematically solving of the optimization problem is most indicated.

From previous studies [1,2], it resulted that constructive systems can be obtained with increasing efficiency, through adhesion to static determination methods that include different economical aspects of design.

For construction design, this way of the optimization problem, it is not always applicable because several objectives and subjective factors must be taken into consideration. Each of these factors has different weight and one of them could have contradictory characteristics and influences. Because of that, methods less compelling are also accepted. In the present phase, rational design tests using simple mathematical models are totally justified.

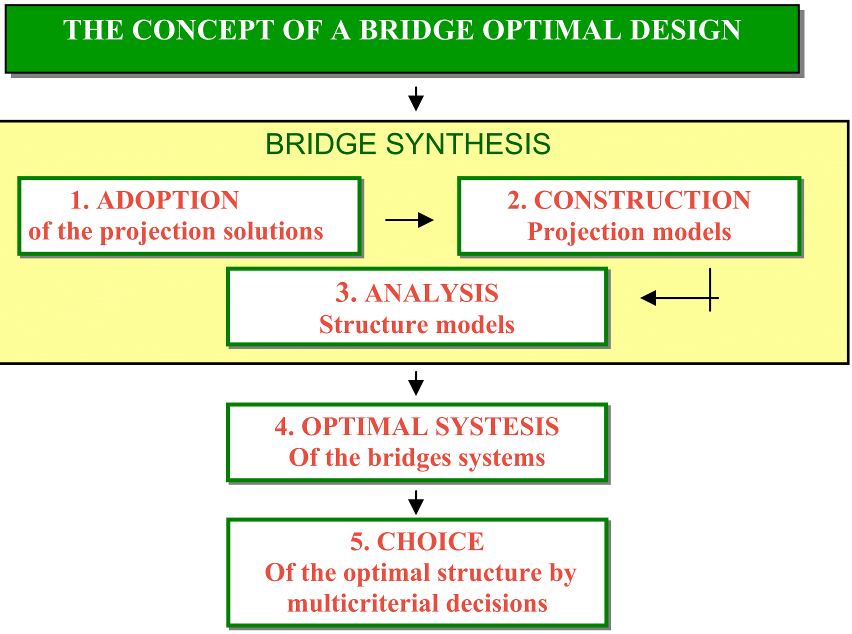

Figure 4. Phases in optimal bridge design

Generally, the design process logistics presents the existence of several phases such as: the problem definition, structure design, bridge structure evaluation, selection and determination of actions. In this rational model, each stage has a clear and right perspective and the opposed connection can reappear between phases. This is due to the effort of rectifying the problem. Therefore a partial overlapping or a fusion between phases can exist during the next stages.

For the optimal design process of a bridge the next logical determination can be observed, see Figure 4. Determination of the optimal solution it is done by putting together the three main processes: the classical design process that follows the phases already presented; the optimal design thinking, obtained by automatic determination programs based on finite elements and the idea of optimal structure choice (from more optimal structures, each of them been placed in its own category); multi-criteria decisions.

From possible multiple variants the optimal one must be finally chosen. The optimization quality of a variant can be determined considering one or several criteria. These bridges criteria are exactly its essential performances, determined by technical characteristics and reliability, maintainability and availability indicators, and the general cost–bridge system and esthetic performances.

3.4. Operational process optimization

The operational phase for a bridge is a cyclic process containing three important processes: the bridge exploitation, bridge maintainability and bridge replacement. From this phase, contrary reactions to the system design phase appear, due to obtaining an operational experience of the bridge. This fact determines new systems creation, with high operational capacities.

A damaged bridge maintainability is defined by the probability that the bridge can be put into exploitation at his operational states in a given time period. The bridge maintainability process will also include a project realization and its applicability. The replacement process leads, in certain limits, to the total realization of a bridge including the design and bridge execution.

Also, the operational phase includes processes which overstep one bridge problem, leading to an analysis of a set of bridges or bridges from a network. This is why it was necessary to imagine an all-inclusive system, named bridge management system (BMS).

BMS contains a series of analytical methods and evaluation procedures of the bridges present state, future state and determination of the best possibility of funds allocation between different activities as: maintenance, rehabilitation and consolidation. BMS facilitates the programs and budgets statement by insuring a process structured on engineering and economical analysis, combining the management, engineering and economical knowledge with the analysis capacity of the modern determination systems.

A BMS contains: a database; degradation and costs models; the optimization module; and actualization functions. In BMS, the bridge behavior and the consequences are studied in several ways:

- The use of deterministic models in order to predict how the bridges will degrade, and the probabilistic modules in order to predict the bridge state levels in time;
- The „top-down" approach. Realization of bridge maintainability program starts with an analysis of the purposes and restrictions on the whole roads network, generating an optimal policy at the network general level. The funds are allocated between alternative projects based on the benefits maximizations or the total cost minimizations. By contrast, the approach „bottom-up" is looking for optimal actions for each bridge or for different standard levels of services. After that the costs are accumulated at the network level, in order to determine the total cost for each standard level that compares with the imposed restrictions;
- Utilization of mechanical models through which the physical process of bridge degradation is mathematically described. It tries to consider the variable multitude that affects this process. The empirical models consider only the most important variables, easier to describe on statistical basis and using data from measurements realized in the past. The judgment models depend on the experience and on human intuition in the case when the empirical or mechanical data are impossible to obtain or inadequate. The main purpose of the BMS applicability at the level of a roads network is to maintain and raise the users' possibility to travel efficiently and safe. From the economical point of view is necessary to determine the users' costs produced by the bridge degradations or the benefits obtained by the users due to the improvements brought to the bridges. A way to determine the connection between the users' costs and the bridge characteristics is given by the service levels standard. Different tonnage restrictions categories will have a corresponding cost for the users. The minimal threshold for these restrictions defines the standard service level. A BMS must evaluate the proportion between the actions of rehabilitation, consolidation and modernization. The first ones are an answer to the deteriorations while the modernization actions are an answer to the users' requests, such as the bridge enlargement.

4. CONCLUSIONS

1. There is a constant preoccupation of the experts, at international level, regarding the rise of the technical bridge level, under the conditions determined from many bridges that are at the exploitation period limit.

2. A certain way to solve this problem is that proposed in the present paper, which consists in defining the bridge engineering area, in connection with the modern sciences: Systems Theory and Engineering, Systems Reliability etc.

3. Three important phases are typical for total bridge realization: design, construction and operational phases that must be unitary treated by putting into function the information flow system and based on information obtained for realization of the bridge processes' optimization.

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