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Geotechnical assessment of the Compressible Soils

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Summary

Compressibility is the property of a soil that permits it to deform under the action of an external compressive load. Loads are primarily static loads that act, or may be assumed to act, vertically downward. Brief mention will be made of the effects of vibration in causing compression. The main concern here relates to the soil property that permits a reduction in thickness (volume) under a load like that applied by the weight of a highway or airfield.

The compressibility of the underlying soil may lead to the settlement of such a structure. These types of soil deposits are often found near the mouths of rivers, along the perimeters of bays, and beneath swamps or lagoons. Soil deposits with high organic content are often found in these low lying types of locations and can be especially troublesome. Since land features in which these troublesome soils are typically found are low lying, they are prone to flooding.

Hence before buildings or roadways can be constructed on such soil deposits, the grade level must be raised by adding compacted fill. However, adding significant amounts of compacted fill puts significant loads on the soil which can cause significant settlements.

For their use as weak foundation requires improvements to measures physical-mechanical properties of the soil. This improvement consists in the use of methods that change the natural structure of the soil on a portion of the thickness of the soil, or the entire thickness, depending on the nature of the terrain, the thickness of the soil and the loads submitted by it. Macropores soils presents numerous fine vertical channels, that makes their irregular porosity to be distributed. The channels are visible to the naked eye and give birth to porosity higher than the normal, called macro porosity. These soils are represented by loess and loess soils.

KEYWORDS: types of soil, compressible, macro pores, porosity, loessoides

1. INTRODUCTION

The behavior of difficult foundation soils is different when compared with regular soils. This difference consists of the fact that they undergo supplementary settlements under constant load, when their humidity increases over the natural wetting. The phenomenon analysis shows that the supplementary wetting of soil on site produces the occurrence of supplementary settlement to material compaction, due to a resettlement of granules and aggregates of which it is formed, in a more compact position. This new arrangement is possible because the soil porosity is much greater than the porosity that would exist at actual geological load to which it is subjected. The soil is characterized by a subcompaction.
In case this phenomenon takes place on a site, that incorporates constructed buildings, takes place a supplementary settlement also due to external loads. In certain cases, these deformations might influence the resistance and behavior of the entire construction.
The above mentioned soil problem is of great interest in the context of a wide diversity of foundation soils and construction types. At this point, the difficult foundation soils can be considered dangerous features for the construction process.

2. DETERMINATION OF SENSITIVE TO WETTING SOILS (SWS) COMPRESIBILITY

For the design on foundation systems in case of macro porous soils, sensitive to wetting, it is necessary to establish several parameters.

2.1. Specific laboratory parameters:

- Oedometer modulus ($E_{oed\ i\text{-}j}$) for different steps (i,j) with the loading, σ, on natural and saturated samples,
- Supplementary settlement indices specific to wetting on different steps ($i_{m\sigma}$) on double oedometer tests,
- Supplementary settlement to wetting indices i_{m300} (for load 300 kPa),
- The structural pressure of soil, σ_0 (figure 1).

The value of structural pressure, σ_0, $i_{m\sigma}$ (expressed in cm/m or %), is defined as the minimum pressure from which the soil presents supplementary settlements at wetting, as a consequence of the reduction of structural bindings and a new arrangement of particles.

$$i_{m\sigma} = \varepsilon_i - \varepsilon_n = 0{,}01 = 1\%$$

This parameters result in oedometer test.

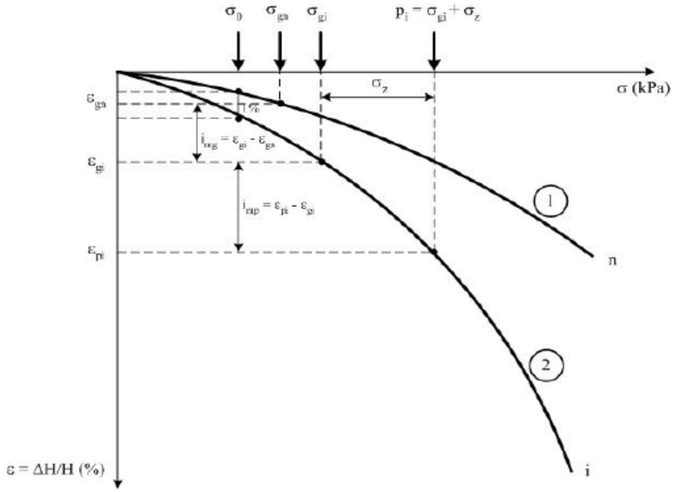

Figure 1. Laboratory oedometer tests. Compression – settlement curves for natural (1) and saturated (2) samples

2.2. In situ specific geotechnical parameters

The in situ geotechnical parameters can be obtained along the plate test until a pressure of at least 300 kPa. The main parameters are:

- ✓ Young modulus, E, can be obtained in natural and saturated state, minimum pressure is 300 kPa,
- ✓ The settlement on natural site, s_n,
- ✓ The settlement on saturated site, s_i.

2.3. The settlement index specific to wetting ($i_{m\sigma}$)

This index represents the difference of specific settlement for a pressure, σ, on the compression - settlement curve. It is obtained along an oedometer test for samples in natural and saturated state [4].

$$i_{m\sigma} = \varepsilon_i - \varepsilon_n \ [\%] \qquad (1.1)$$

Where:

ε_i - specific saturated deformation of soil sample,

ε_n – specific natural deformation of soil sample.

3. SOIL CALCULATION AT NORMAL LIMIT STATE EXPLOITATION

The value of total supplementary settlement by wetting (S) can be determined using the below relation. For the evaluation of supplementary settlement to wetting will be taken into consideration the settlements under the action of geological pressure (I_{mg}) and external pressure (I_{mp}) according to the difficult foundation soil group (A or B).

$$S = I_{mg} + I_{mp} \quad (1.2)$$

Where:
S – total supplementary settlement by wetting,
I_{mg} – supplementary settlement by wetting, under geological pressure,
I_{mp} – supplementary settlement by wetting, under the load transmitted by building foundations.

3.1. Supplementary settlement by wetting under geological pressure (I_{mg})

The determination of supplementary settlement is based on the elementary layers settlements summing calculation method [5].
The supplementary settlement by wetting is being determined on the entire soil layer thickness with the condition $i_{m3} \geq 2$ % with the relation:

$$I_{mg} = \sum_{i=1}^{n} i_{mg_i} hi \quad (1.3)$$

Where:
i_{mg} – represents the index of specific supplementary settlement by wetting for an elementary layer i, under the action of soil geological pressure. It can be obtained using a double oedometer test.

$$i_{mg_i} = \varepsilon_{gi} - \varepsilon_{gn} \quad (1.4)$$

Where:
ε_{gi} - specific deformation for saturated soil under the action of saturated geological pressure, σ_{gi} . The pressure σ_{gi} is calculated with saturated unit weight, γ_{sat},
ε_{gn} - specific deformation for natural soil under the action of natural geological pressure, σ_{gn}. The pressure σ_{gn} is calculated with natural unit weight, γ,
n – the number of elementary calculation layers,
h_i – elementary layers thickness.

According to the value of I_{mg} the soils can be classified as:

- ✓ Group A – represents soil types which at wetting under the geological pressure presents supplementary settlements where: $I_{mg} \leq 5$ cm,

- ✓ Group B - represents soil types which at wetting under the geological pressure presents supplementary settlements where $I_{mg} > 5$ cm.

3.2. Supplementary settlement by wetting under external foundation pressure (I_{mp})

The supplementary settlement by wetting under external foundation pressure is being determined from the foundation depth, D_f, with the relation:

$$I_{mp} = \sum_{i=1}^{n} i_{mp_i} \cdot h_i \cdot m \qquad (1.5)$$

Where:
i_{mp} - supplementary specific settlement by wetting of the soil layer of *i* order,
h_i - soil layer thickness of in order is established by taking into account the lithological profile and it must not exceed 1 m,
m - working condition coefficient,
D_f - foundation depth.

The value i_{mp} is determined with oedometer test and computed based on compression – settlement curves. It allows the determination of deformations in natural and saturated state, respectively:

$$i_{mp} = \varepsilon_{pi} - \varepsilon_{pn} \qquad (1.6)$$

Where:
ε_{pi} - specific settlement of the saturated sample for the pressure $p_i = g_i + p_{zi}$,
ε_{pn} - specific settlement of the natural sample for the pressure $p_n = g_n + p_{zi}$,
g_i and g_n - geological pressures of soils in saturated state (g_i) and natural state (g_n) respectively. These values correspond to the depth, z_i, considered in the center of the elementary layer of *i* order.
p_{zi} – the total pressure under foundation at depth z_i.
σ_z – the normal stress under the action of external foundation pressure (p_{net}).

4. CONCLUSIONS

Following the extended construction process, all around the world, Romania including, construction engineers found themselves in the context of building in difficult foundation conditions.
The expansive or collapsible soils may cause high differential phenomena in structures resistance and behavior, as a result of excessive deformation or

settlement. The present paper highlighted the way of determining supplementary settlement to wetting according to the current norms.
Geotechnical engineers must be able to identify difficult soils when these might be encountered on a specific site. Although it is not possible to solve the wide diversity of problems caused by difficult foundation soils, preventive measures can be taken in order to reduce the potential damage of structures built on these sites.

References

1. Romanian Doc. NP125/2010, Normativ privind fundarea construcțiilor pe pământuri sensibile la umezire.
2. Romanian Doc. NP 112/2004, Normativ privind proiectarea structurilor de fundare directă.
3. Nicuță Ana, Geotechnics, Editura Matei Teiu Botez, 2008.
4. Silion .T ,Răileanu P, Muşat V., Ana Nicuță, Plătică D." Fundații în condiții speciale". Institutul Politehnic Iaşi,1991.
5. STAS 8942/1-1989 Determinarea compresibilității pământurilor prin încercarea edometrică.
6. SR EN 1997- 2:2007/NB:2009, Eurocod7:Proiectarea geotehnică. Partea 2: Investigarea şi încercarea terenului. Anexă națională.

Virtual Representation of the Area of the Bridge – Obstacle – Road System

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Summary

The present paper is divided into three distinct sections. First part defines the bridge – obstacle – road high complexity system, from the point of view of the major subsystems that compose it, and also refers to a fundamental problem of the theory of systems – system synthesis, with great applications in the field of technical systems, where the key issue is the development of systems with well-defined functionality and performance proprieties. In this context as an application of system synthesis in the field of bridge engineering will be cited bridge design and construction.

The second section of this paper analyses the "bridge –obstacle – road system area", defining this concept as an element developed on at least two directions: the longitudinal direction of the bridge (the road direction) and the direction of the crossed obstacle (in this case river direction).

The last section of the present paper presents a virtual simulation of the bridge – obstacle – road system area, analysing this concept as an application of the system theory into bridge engineering field.

KEYWORDS: bridge, system, bridge - obstacle – road system (B.O.R. system), bridge area, B.O.R. system area, virtual simulation, virtual construction.

1. THE "BRIDGE–OBSTACLE–ROAD" SYSTEM SYNTHESIS (DESIGN)

1.1. The "Bridge –Obstacle – Road" High Complexity System

Starting with the **bridge** definition given by 5626/92 standard [1] – *"a construction that supports a communication way over an obstacle, leaving a free space for the crossed obstacle continuity"* and bringing in addition some information about the communication way (road, railroad, piping system, etc.), about the crossed obstacle (valley, a stream, another communication way) we can distinguish the following elements that compose the bridge structure as an whole:

- The bridge – as an construction;
- The road – as an communication way;
- The crossed obstacle – a river stream.

Analysing the **system** definition [2] – "*a set of interdependent elements forming an organized whole, that puts order into a system of though, that governs the classification of material in a field of science or makes an application to operate as intended*" or in other words „*a system is identifed whenever is revealed a relationship between at least two elements or items*" [3] – we could identify the "bridge obstacle – road" system, a system with closed system structure (fig1).

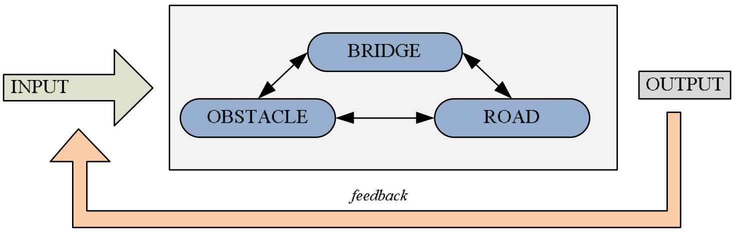

Fig.1. "Bridge – obstacle – road" system. The structure of "B.O.R" system

The system is caracterized by the presence of the reverse link - **feedback –** between at least one output and one input, so the output depends on input but also could partialy or totally change it.

The complexity of the "B.O.R" system can be analyzed from at least two points of view:

- The complexity of the bridge – obstacle – road system is revealed by the complexity of the bridge area, defined by the interactions between the bridge structure and the following: the communication way that crosses the obstacle, the crossed obstacle and the environment (fig.2).

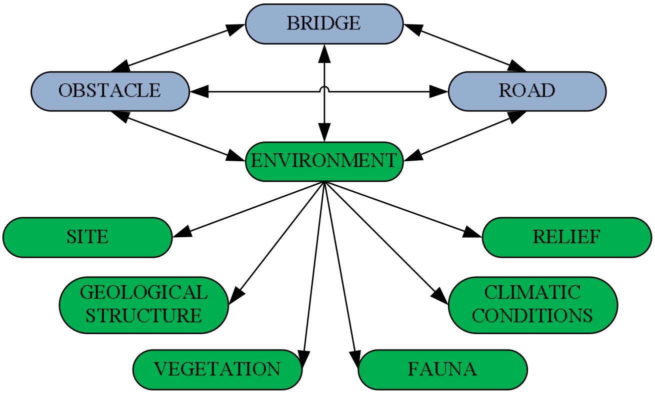

Fig.2.The Interaction between "B.O.R" system and the environment

- The second way to express the "B.O.R" system complexity refers to the cycles that the system is passing through, beginning with the design phase and continuing with the construction phase, the operational phase and ending at a time by replacing (post –use). If to the above mentioned cycles are added the system evaluation phase, the system management phase and the decision-making system, the concept of full realization of "B.O.R" system is defined (fig. 3).

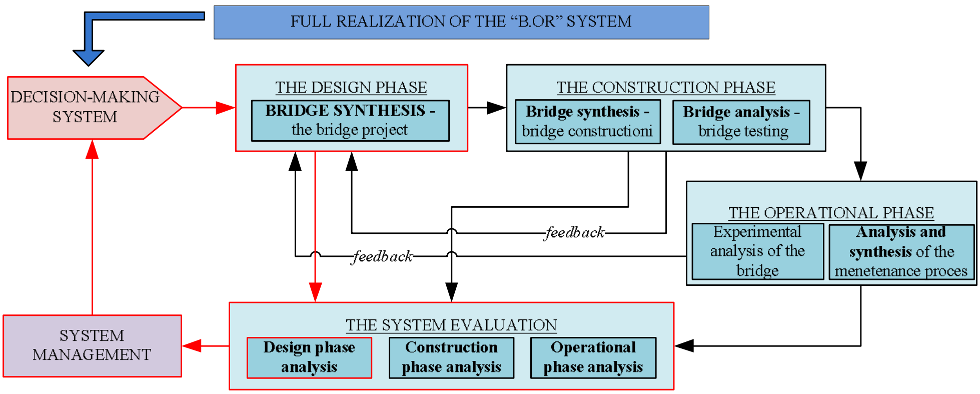

Fig.3.The full realization of the "B.O.R" system – bridge engineering cycles

The full realization of the bridge is a process which includes many aspects of economic, technological, social and environmental nature. To solve this problem it is proposed to call the systems engineering concepts and methods in conjunction with those of bridge engineering.

The systemic approach of the bridge, by introducing the concept of full realization of the bridge-obstacle-road system allows the application of scientific methods in order to integrate the definition, the designing, the planning, the development, the implementation and the evaluation of the system as a whole.

This concept tries to optimize, the design, the execution (construction of the bridge), the maintenance (during the operational phase) phases so that the "BOR" system to be optimal in terms of cost-benefit throughout its existence.

Thus we can conclude by the fact that the full realization of the bridge-obstacle-road system naturally takes the fundamental problems of systems theory: analysis, synthesis and systems management.

1.2. Bridge – Obstacle – Road System Synthesis

Any problem involving the concept of system is based on the existence, identified or presumed, of a system with a structure and function that are some information about. Generally this information is not complete; the knowledge of the system to a certain level in order to enable some conscious actions on it remains a task that is based on the original information.

The development of the mathematical techniques and methods of this theory reflects the tendency of systems theory to address a single issue ever wider.

The current status of systems theory allows us to distinguish several types of problems:

- System analysis,
- System synthesis,
- System management.

If the system analysis determines the structure and the functionality of a system from its presumed existence, the synthesis answers to the problem of building the system in its abstract (virtual) form or physical form with a certain functionality and some certain proprities that must be achieved in order to construct the ssysytem in its physical form.

Synthesis can be geared towards the achieving of a specific performance of the system -by performance meaning a certain relationship between inputs, states and outputs, which is not its own, but are required to take the maximum or minimum values.

System synthesis has great applications in the field of technical systems, where the key issue is the development of systems with well-defined functionality and performance proprieties. In this context as an application of system synthesis in the field of bridge engineering will be cited bridge design phase and bridge construction phase.

In view of the above, the design phase can be modelled, using systems engineering concepts and specific methods of systems theory, through a closed system (fig.4).

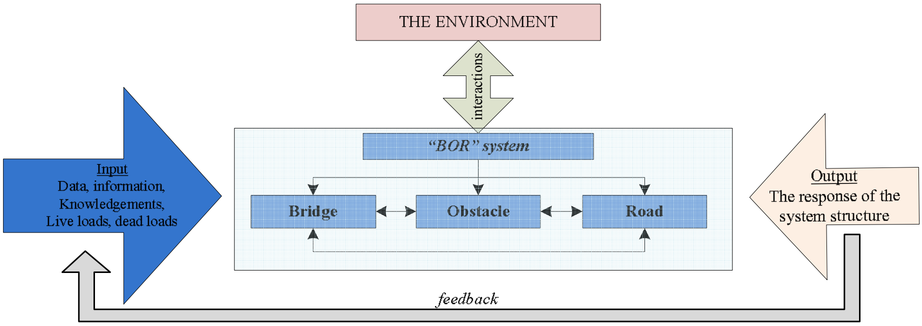

Fig.4. "BOR" system synthesis (design)

The system uses a negative feedback because it makes an inspection of the output on the entry, in order to bring the output to a level required by the structural and functional performance criteria.

In conclusion, "BOR" high complexity system synthesis (design) is a process that requires knowledge of inputs and outputs in order to determine the system structure.

In other words, the design (synthesis) of a new bridge, as a phase of the full realization of the "BOR" system, can be considered as a phase of virtual construction of the system [4], in both scientifically and technically points of view (fig.5). In this context we can move to the other two fundamental problems of systems theory: system analysis and system management.

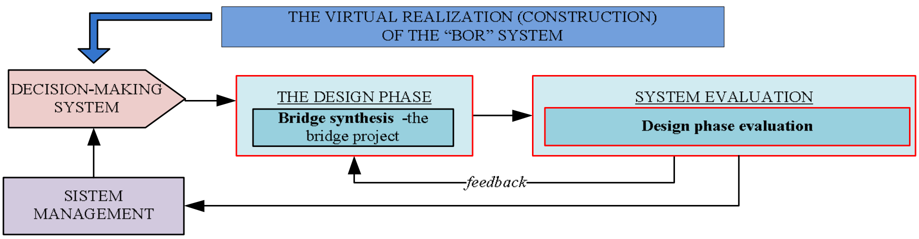

Fig.5.The virtual realization (construction) phase of the "BOR" system

2. AREA OF THE "BRIDGE–OBSTACLE–ROAD" SYSTEM

2.1. Mention of the "bridge area" concept in specialized literature

The legislation actually used in our country uses in some cases the concept, the term, of "bridge area" without explicitly defining it.

For example the order no. 48 /27.01.1998 regarding the approval of the placement and operation of the ballast-pits in bridge or road area [5], uses the term of **bridge area** in the title and in subsection 3.2, which states: “a new ballast pit must be placed at not less than 1km upstream and not less than 2km downstream from the bridge area”. This prohibition is explained by the fact that the exploitation of natural aggregates leads to changes of the river bed level which in some cases can cause the crumbling of roads structure, settlements or rotations of the pile foundations and abutments, leading to disruptions and restrictions of the traffic.

Another example, also in the same direction, is the CD 99-2001 regarding the technical instructions for repair and maintenance of concrete, reinforced concrete, prestressed concrete and stone masonry road bridges and culverts [6], mentioning in subsection 2.7.3 “the major and minor maintenance works of the riverbed of the watercourse, in the **bridge area**, are as follows:

- The cleaning vegetation on an area of approx. 100 m downstream and upstream of the bridge axis. Will be performed quarterly. For very long bridges, cleaning vegetation will be made on the minimum length equal to the length of the bridge, downstream and upstream of the bridge;
- The removing of the solid deposits transported in the riverbed, in the **bridge area**. Will be performed quarterly…”

Also, the same document is referring to the bridge area in the subsection 3.7.3 as follows:” the riverbed arrangement in the bridge area, as appropriate: bottom sills, bank revetments or unclogging the riverbed…”

In addition, it can be quote PD95/2002 [7], which contains the **bridge area** expression:” the scouring of the river bed in **bridge area**, for the rivers with erodible beds or banks, are calculated in order to determine the depth of substructure foundations, river banks defense works as well as the length of the bridge”.

It could be noted that in all cases mentioned above, the use of the term bridge area is made in relation to the crossed obstacle, here the river stream.

In a final category of technical regulations that use the word bridge in situations where the use of the term bridge area would better explain the concepts and the terms contained in context, could be cited the AND522-2006 [8] in which the establishment of a technical condition of a bridge is taking into account the channel quality indicators, bank protection, access ramps which, naturally, are not part of the bridge structure.

2.2. The defining of “bridge area” concept

In order to define the bridge area concept it is necessary to nominate two different directions: the longitudinal direction of the bridge and the direction of the crossed obstacle (stream, railway, valley and floodplain developed land area).

The bridge area must be composed by at least the following elements:

- The bridge structure (road bridge, viaduct and access ramps, combined overlapped bridge, combined juxtaposed bridge, walkway overpass);
- The access ramps and the embankments behind the abutments;
- The safety areas;
- The protection areas.

The safety areas and the protection areas in the longitudinal direction of the bridge could be defined in the same way as the safety and protection areas of the public roads [9].

The safety areas are land surfaces situated on both sides of the road footprint, designed exclusively for road signs, for road plantation and other purposes related to the maintenance and operation of road traffic safety or for the protection of the private properties from the vicinity of the road. Safety areas also include the land areas that are designed to ensure visibility in curves and intersections, and the areas occupied by the consolidation works of the road.

Outside cities, the minimum safety areas of the road in both the current way and alignment are provided as follows:

- 1.50 m from the outer edge of ditches, for roads that are located on the ground level;
- 2.00 m from the foot slope for embankment roads;
- 3.00 m from the top of the slope, for the roads in cut to 5.00 m in height including;
- 5.00 m from the top of the slope, for the roads in cut greater than 5.00 m in height;

The protection areas are the surfaces of land located on both sides of the safety areas that are necessary for the protection and the future development of the road structure. The limits of the protection areas are established considering the category of the road and they are measured as a distance between the road axis and the exterior edge of the road area.

The **safety area** on the direction of the crossed obstacle could be defined as follows:

- For the river stream the safety area could be established as a surface determined by the width of the riverbed and the length of the longitudinal dikes from the upstream of the bridge, and the the length of the construction of bottom sill, downstream of the bridge. In the case of river streams without landscaping works the safety area is determined by the surface created by the width of the riverbed and a length of 100m upstream and 100m downstream of the bridge axis (for usual bridges). For the large bridges the safety area is defined as the surface obtained from the riverbed width and an equal length of the bridge span measured in upstream and downstream of the bridge axis;

- In the case of the flood areas or landscaped areas the safety area will be determined in the same way as for river streams, 100 meters upstream and downstream of the bridge axis;

The protection areas on the direction of the crossed obstacle, in the river stream cases, could be established as the surface created by the riverbed width including the major riverbed and and the length of 1 km, upstream of the bridge and the length of 2 km, downstream of the bridge [5].

Once the bridge area term is stated it becomes a basic concept in bridge engineering, to which the specialist will refer to determine technical and aesthetical solutions or environmental effects analysis.

3. VIRTUAL REPRESENTATION (CONSTRUCTION) OF THE BRIDGE – OBSTACLE –ROAD SYSTEM AREA

3.1. The system management process – simulation process

A major problem in systems theory is the systems management, which subordinates the both system analysis and system synthesis. The ways of solving this problem include the specialists' expertise, the simulation and optimization of "BOR" system. The last two issues are possible especially when using electronic computer.

Simulation is a technique of carrying out experiments with digital computer, which involves the construction of mathematical and logical models by which may be described the behavior of a real system over a longer or shorter period of time. From the scientific point of view simulation means a simplified and artificial reproduction of a phenomenon that occurs in nature.

The need of simulation process is ordered by the fact that real systems cannot be studied directly because of their complexity (large number of input and output variables, large number of possible states, the complexity of functions).

There are several methods of simulation:

- Physical simulation of systems achieved by reducing the scale of the real system;
- Simulation of the physical system based on analogies with mechanical systems;
- Numerical simulations, the system or process under study is mapped to a mathematical model that should be easy to handle and represent system behavior in the most rigorous way.

The process of simulation has the following advantages:

- Through the formulation and testing of models can be systematically collected reliable data and often suggestive data;
- The simulation process is generally highlighting those variables that have a particular significance for the study of real phenomena, putting into light the links between these variables;
- simulation is much cheaper than many other forms of experimentation;
- Simulation allows the intuition of the real phenomena, and therefore is an instructive process;
- In most cases, the simulation allows control over the time, so events that may take a long period of time to occur can be studied in a few minutes;
- The simulation process is a secure process because it can be interfered in model as necessary, without causing disruption on the development of the phenomenon of the real model.

The objectives of the simulation are mentioned as follows (fig.6):

- The checking of a uncertain solution obtained by analytical methods;
- The collection of conclusive and suggestive data;
- The optimization of some elements of the bridge or bridge structure entirely.

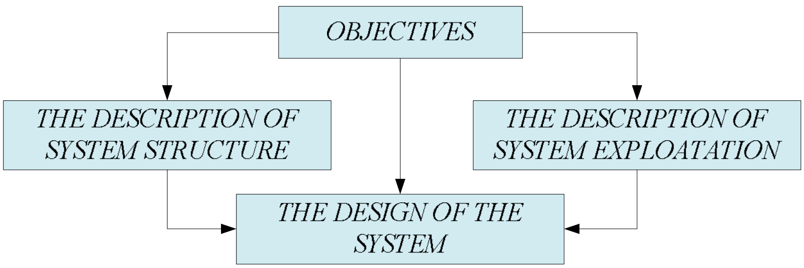

Fig.6. The objectives of system simulation

The simulation always involves the use of a model. Therefore by simulation we understand a technique to achieve numerical experiments using computer and involving the use of logical or mathematical models that describe the behavior of the real system in a period of time.

Currently, simulation is used for both to design a new system and to analyze the existing ones in order to improve their performance.

3.2. Virtual Representation of the Area of the "BOR" System

In this section it will be presented the simulation of "bridge – obstacle – road" system area, considering both the designing phases and the construction phases of the system.

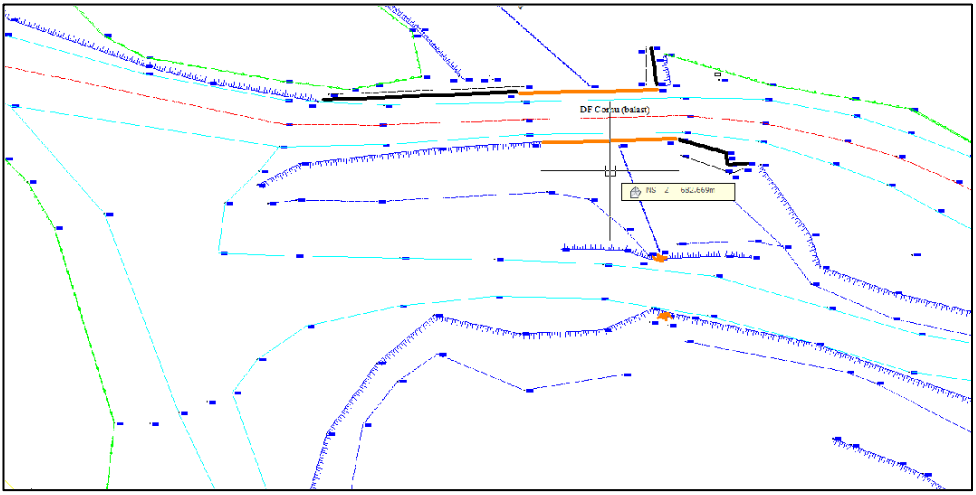

Fig.7. The topographic survey of the bridge site.

Starting with the data gathered by conducting studies in the bridge site, topographic surveys (fig.7), geological studies and completing these studies with the hydrological parameters of the crossed river, both the existing ground level surface (fig.8) and bridge subsystem structure (fig.9) could be modeled (simulated).

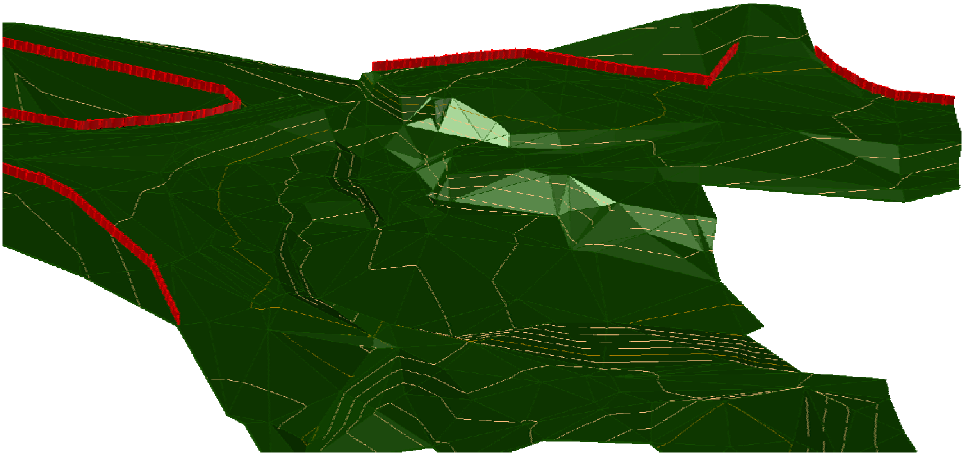

Fig.8. The existing ground level surface simulation.

The ground level surface is meshed using topographic triangulation networks according to the STEREO70 projection system. The surface also contains the private properties limits which must not be trespassed during the project implementation phase (construction phase).

Using as an input data the Romanian norms and standards of bridge design, the bridge subsystem structure was simulated (fig.9). This stage highlighted the importance of the obstacle subsystem whose hydraulic parameters imposed both length and height of the bridge.

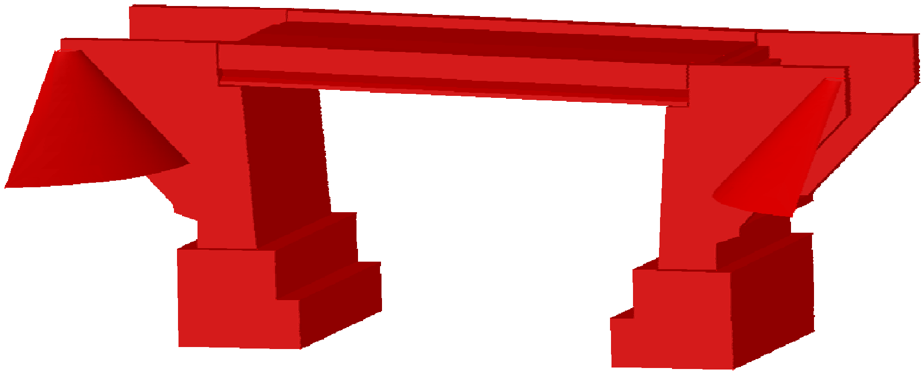

Fig.9. The bridge subsystem structure.

The importance of the crossed obstacle subsystem, a river stream in this case, is revealed again by the fact that his hydraulic parameters imposed the plan settlement of the bridge which in turn imposed the road centerline course (fig.10).

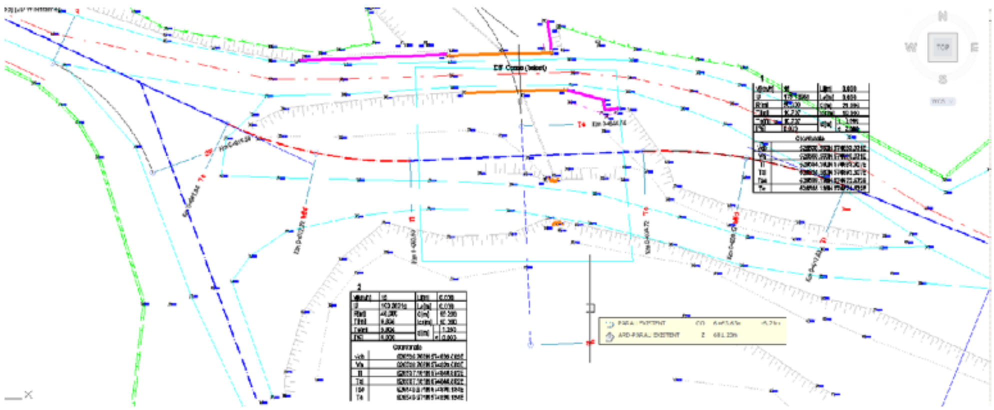

Fig.10. The road centerline course.

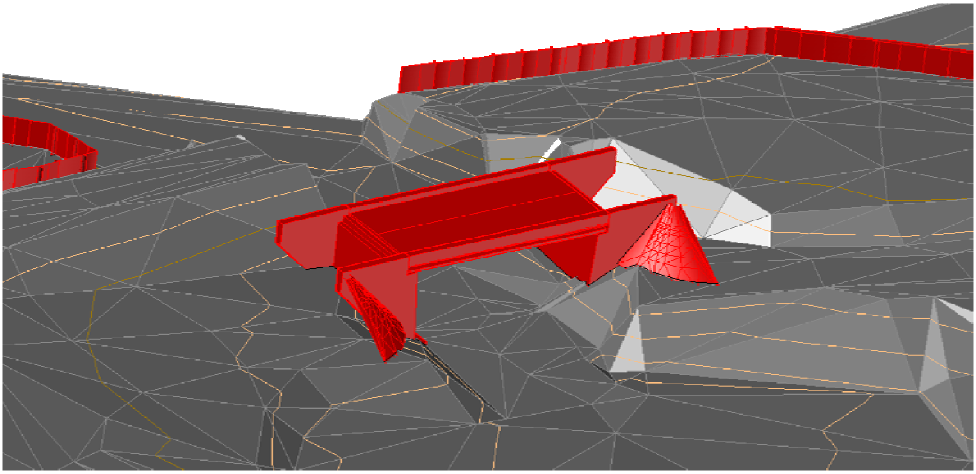

Fig.11. The simulation of the bridge construction on site (ground level surface and the bridge subsystem).

Using as an input data the Romanian norms and standards of roads design, the road subsystem structure was simulated (fig.12,13).

The simulation is conducted considering the vertical designing of the road centerline (vertical grade), the horizontal design of the road (curves, super elevations, road width expansion, etc.), and the design of the road cross section.

The horizontal design of the road also contains information about the visibility curves, simulating this way the traffic conditions.

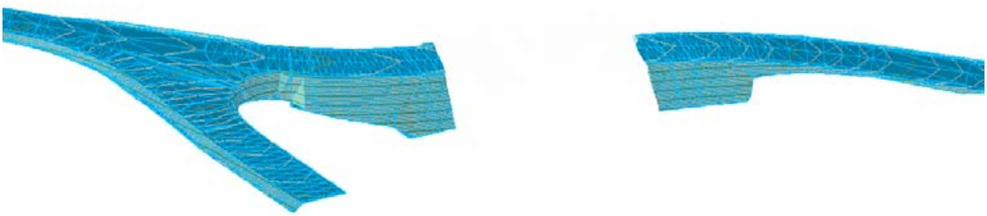

Fig.12. The access ramps as a standalone subsystems – lateral view

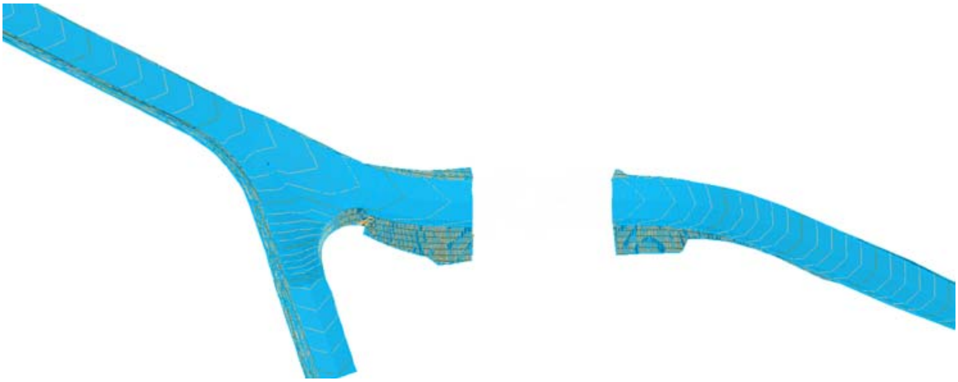

Fig.13. The access ramps as a standalone subsystems – plan view

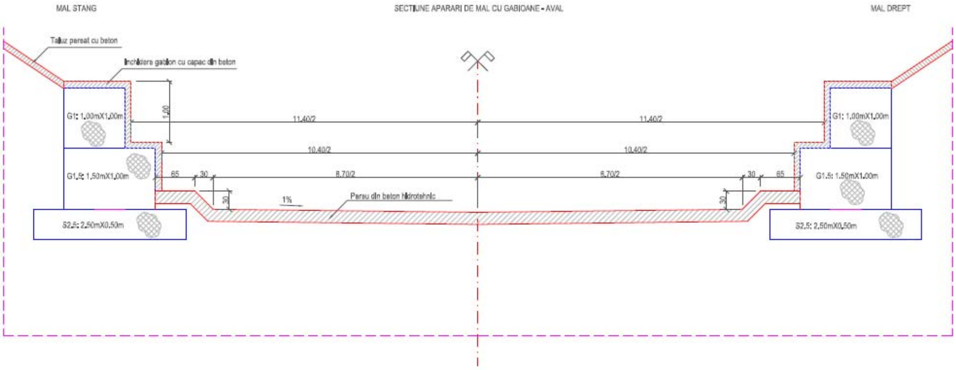

Fig.14. The designed riverbed protection works (downstream side of the bridge)

According to PD95/2002 [7] norm, regarding the hydraulically design of bridges and culverts, were designed the river banks defense works and riverbed protection works (fig14, 15).

In this case the riverbed protection works covers three lengths of bridge span: one bridge span downstream and two bridge spans upstream, measured from the road centerline.

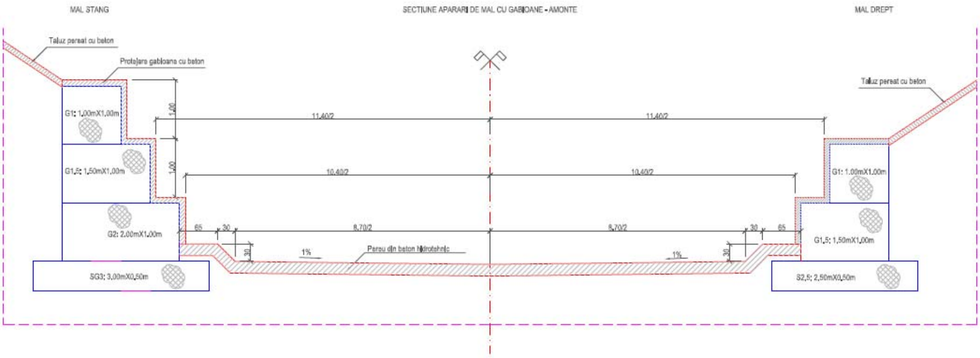

Fig.15. The designed riverbed protection works (upstream side of the bridge)

The designing of the riverbed protection works, hydraulic concrete pitching on a depth of 15cm and gabions for the river banks defense works, both in longitudinal direction of the river (regarding the river centerline grade) and in transversal direction of the river (cross section) enables the simulation of the crossed obstacle.

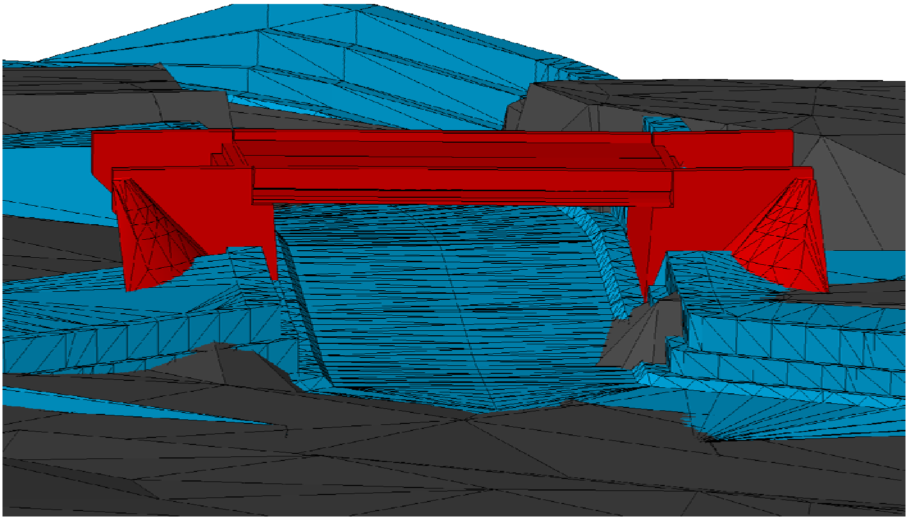

Fig. 16. The simulation of the designed river surface (crossed obstacle and bridge subsystems)

Finely by considering the bridge, the approach ramps and the crossed obstacle – the river as the subsystems that compose the “bridge – obstacle – road“ system it is possible to simulate the “BOR” system area.

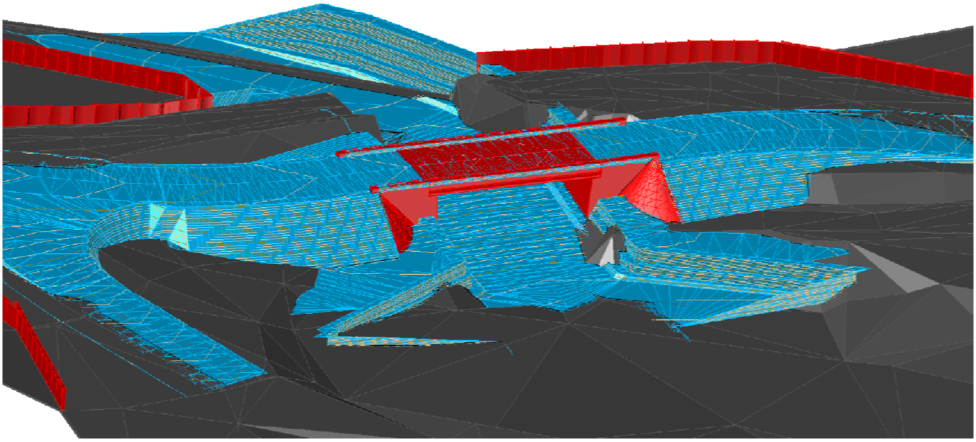

Fig.17. The simulation (virtual representation) of the “BOR” system area

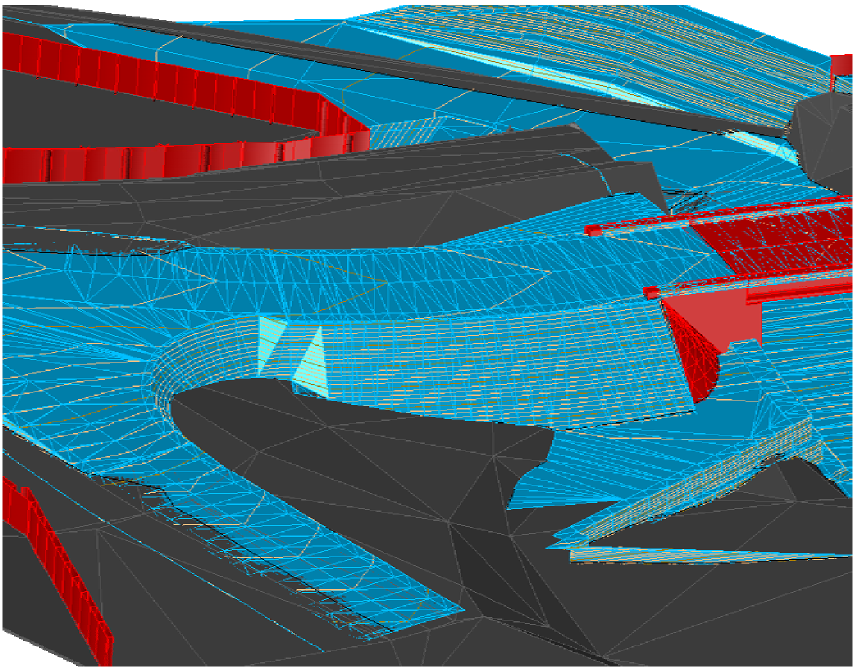

Fig.18. The virtual representation of the right bank approach ramp.

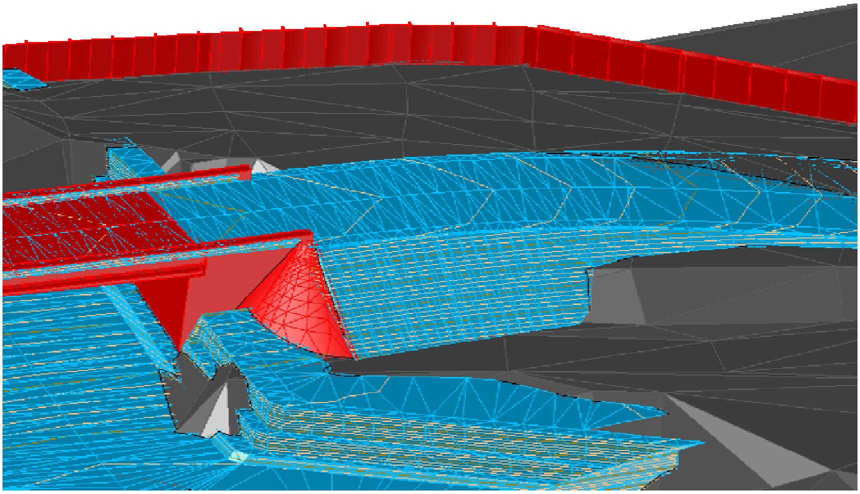

Fig.19. The virtual representation of the left bank approach ramp.

The objectives that were achieved by the simulation of the area of the "bridge – obstacle – road" system are as mentioned:

- The optimization of bridge settlement and road centerline horizontal design;
- Plan plotting of the area of the bridge – obstacle – road system (fig.1.20) that could be included in the plans list of the project;

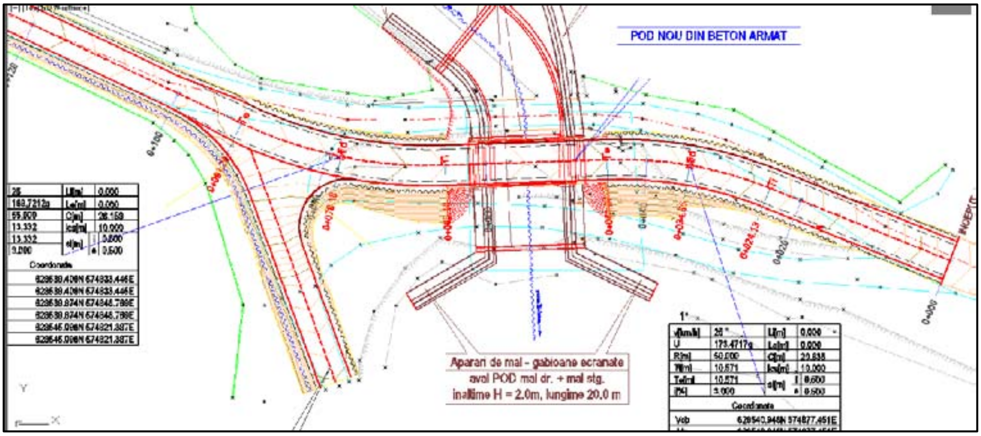

Fig.20. the plan plot of the "BOR" system area

- The collection of conclusive and suggestive data regarding the volume of earth works (cut and fill volume) , the volume of material used for the road structure construction, the volume of material used for the bridge structure construction, the volume of material used for the riverbed and riverbank protection works which ensured an exact bill of quantities reporting;

References

1. STAS 5626-92 Poduri.Termonilogie.
2. Edmond Nicolau, Al. Popovici, Inițiere în cibernetica sistemelor continue, Ed. Tehnică, Bucureşti, 1972.
3. Gheorghe, V.A., Ingineria sistemelor. Metodici şi tehnici de calcul, Editura Academiei Române, Bucureşti, 1979.
4. Cristian Blejeru, „Evaluare" în procesul de realizare a studiului de fezabilitate (prefezabilitate) a unui pod metallic, Creații Universitare2010, Al III-lea Simpozion Național, 2010.
5. Ordin 48/27.01.1998 - pentru aprobarea Normelor privind amplasarea şi exploatarea balastierelor din zona drumului şi a podurilor, Monitorul Oficial nr.138bis din 6 aprilie 1998.
6. CD 99-2001 - Instrucțiuni tehnice privind repararea şi întreținerea podurilor şi podețelor de şosea din beton, beton armat, beton precomprimat şi zidărie de piatră.
7. PD 95-2002 - Normativ privind proiectarea hidraulică a podurilor şi podețelor.
8. AND 522-2006 - Instrucțiuni tehnice pentru stabilirea stării tehnice a unui pod.
9. Ordonanța 43 din 28 august 1997 actualizată în 2003 – privind regimul drumurilor.

Capital repair of bridge over Romanian sluice at Iron Gate I

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Summary

The paper presents the repairs stages to the access bridge that serves the biggest hydro-power plant over the Danube River, making the passage over the canal lock. The bridge has been used for 40 years, a period in which no significant repairing works have been done. The total length is of 260.85 m distributed over 9 spans. The bridge comprises three curves with rays of 45 and 30 m. The structure is made of three frames, the connection between them being made by two independent decks.

At this moment the repairs work is 95% finished. Work has kept a year and a half during which time have been repaired on the bridge access stairs, beaks the supporting and path.

KEYWORDS: rehabilitation, concrete bridge, structural health monitoring, bridge management

1. INTRODUCTION

The bridge is located within the Iron Gates hydroelectric complex, near National Road 6, which connects Drobeta Turnu Severin and Orsova.

The bridge is located within the Iron Gates hydroelectric complex, near National Road 6, which connects Drobeta Turnu Severin and Orsova.

The bridge ensure access from the national road inside the PF I.

The bridge has two lanes with a total width of 10.00 m and is intended for use on local roads of exploitation characteristic traffic. Pedestrian traffic is done on two sidewalk width of 0.90 m and 1.30 m, inside pavements are mounted electrical and technical installations.

In 2008 S.C. Hidroelectrica S.A. has analyzed the status of the bridge, and established like priority bridge rehabilitation over the head of Romanian downstream of the lock, placed on the operating way SHEN - Iron Gate I.

The bridge has the following characteristics:

Bridge Type:

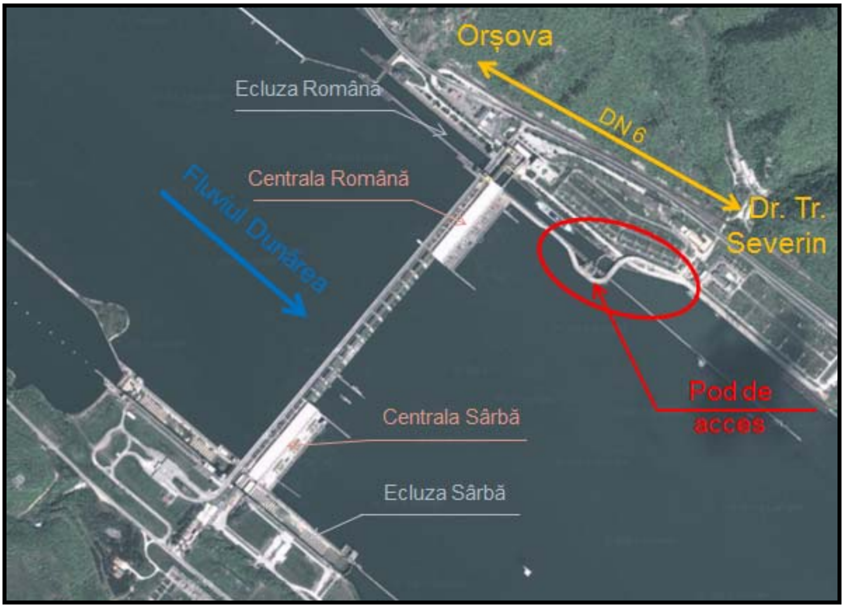

Fig. 1 – Displacement of bridge

- - by static scheme: box frames console and joints
- - by structure: Gerber system
- - the mode of execution: monolith

The number of openings and their length: 9 openings

25,35+32,00+32,00+37,10+18,75+29,45+28,85+30,00+27,35

Total length of bridge: 260.85 m

The carriageway width: 10.0 m

Free to low-water line height: approx. 10.5 m above the max

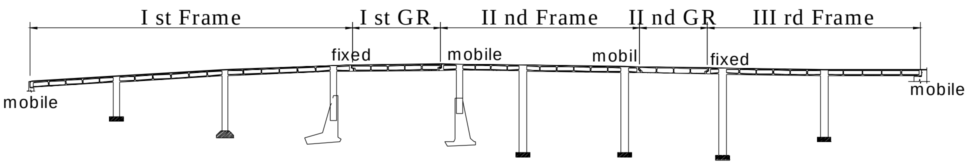

Fig. 2 – Longitudinal view of bridge

The resistance structure of bridge is composed of three independent frames with cast circular monolith pillars and box beams. The main systems support two simply supported plates, making a total static type Gerber system.

Deck superstructure, with box section and inclined walls has a constant height of 2.05 m measured in the center of roadway. The bottom width of box is constant along the length of the bridge, with a value of 5.00 m, and on top with is 6.30 m. The vertical walls of the box have thicknesses of 0.70 m for boxes from nearby nodes to shear force resist. For sub-sections by node vertical thickness is 0.50 m. For other boxes from the mid the thickness walls is 0.30 m. Boxes are separated by diaphragms, reinforcing the role of the section box. Diaphragms thickness is 0.25 m.

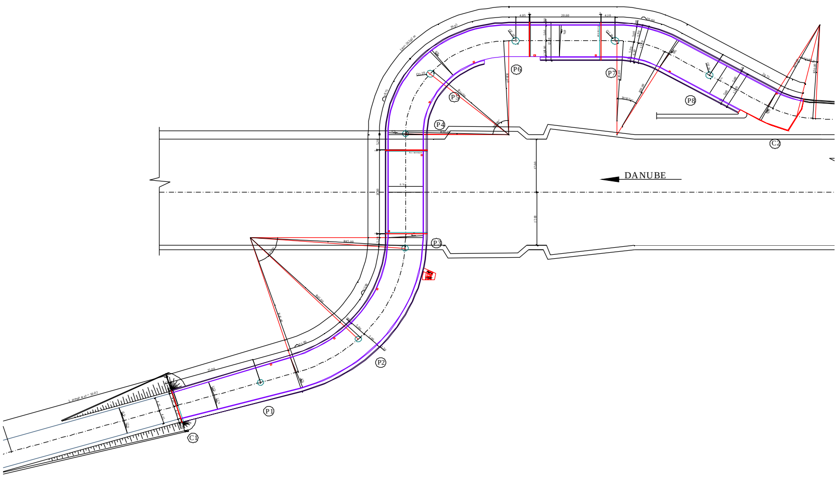

Fig. 3 – Plan view of bridge

2. STAGES OF WORK

The main problem of the superstructure of this bridge is manifested by advanced degradation of resistance structure as evidenced by the displacement of concrete and extensive corrosion and powerful especially in the closed area on the console, where you can see the deck lowered. Lowering the independent deck can have as main cause failure of consoles. This process of advanced degradation, of the concrete and steel of these main elements are because of water seepage through the expansion joints

Due to water infiltration into deck expansion joints produce concrete degradation occurred independently and excessive corrosion of the reinforcement, which over time has produced a compaction of the deck to the bearing bracket

Works have focused on eliminating defects and degradation, with the following order of priorities

- - Repair of the supporting beaks of frames and independent decks.
- - Rehabilitation of the bridge underside
- - Repair of infrastructure
- - Replacement of the bridge path
- - Repair of the road connection with the bridge

2.1. Repair of the supporting beaks

Work began by assembling scaffolding, demolition of damaged concrete, clean of reinforcement and concreting

Fig. 4 – Degradation of concrete

Fig. 5 – Concrete demolition

Repair the supporting beaks was done to increase load capacity.

After repairs have been made the preparations for independent deck lift to bring in the correct position thrust bearing.

Because the bridge deck was not horizontally, making a plan was needed to pose.

During the entire period of the works had to be able to perform road traffic on the bridge

Lifting structure form a yoke which supports with one end in the middle of independent beam and the other end on the end cross pieces of the frame.

Fig. 6 – Reinforcement protection

Fig. 7 - Concreting

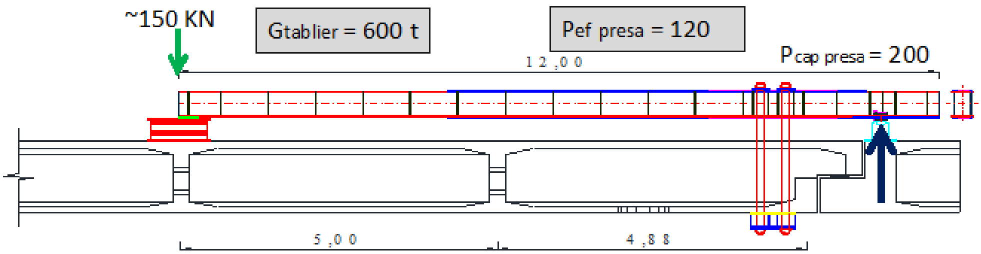

Fig. 8 – Longitudinal displacement of lighting structure

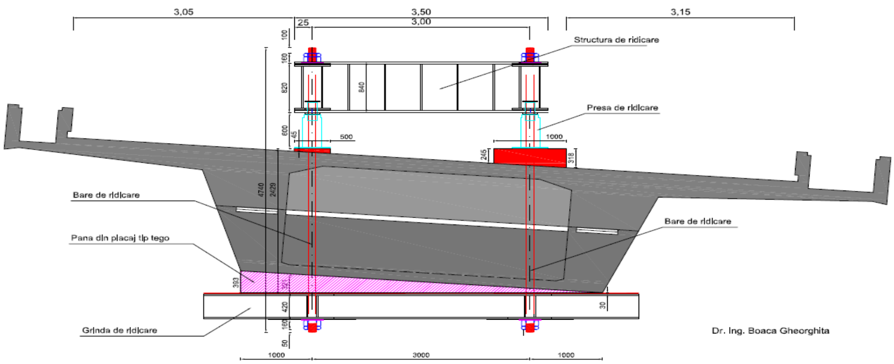

Fig. 9 – Transversal displacement of lifting structure

Before lifting have been blocked the expansion joints of the structure by inserting metal plates to prevent movement in the longitudinal direction. Raising the superstructure was made with two 200-ton hydraulic jacks equipped with safety device and comparators watches.

Fig. 10 – Lifting the independent deck

Lifting the structure was done in several stages. First lifting of the structure was 5 cm, then hydraulic jacks have been blocked. After the first lifting was verified longitudinal movement of the deck.

During lifting the traffic on bridge was stopped and after it totals were performed ramps so that vehicles to be successful.

Initially it was predicted to give a lifting of 20 cm, but it was necessary a lifting of only 5 cm.

Fig. 11 – Lifting the independent deck

The main objective of lifting was to cleaning the bearing and bringing them to the correct position. Cleaning of the metal parts was done by blasting process that has brought real benefits. Protective equipment made subsequent to paint the metal parts.

The final of the lifting was to put on the correct position of bearing.

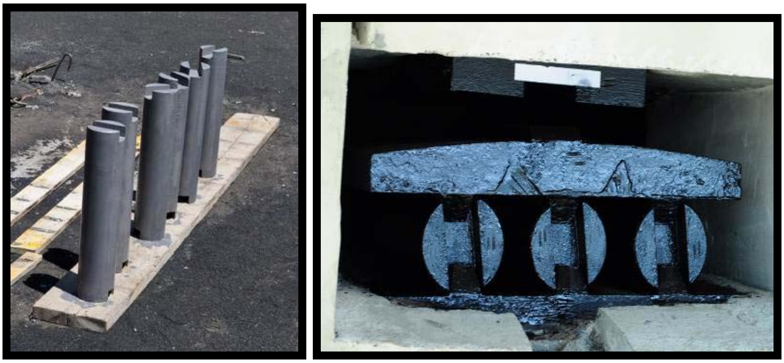

Fig. 12 – Bearing device protection

2.2. Rehabilitation of the bridge underside

On the underside of the deck were necessary local concrete repair. The only inconvenient was the large area which had to be protected from corrosion.

Fig. 13 – Reparation of the underside deck

2.3. Repair of infrastructure

The bridge infrastructure was in relatively good condition with some damage and deficiencies in the supporting abutment at their banquets. The supporting seat of abutment C1 could see earth deposits, erosion and efflorescence of concrete, poor quality and especially elevation bearings.

2.4. Replacement of the bridge path

The defects and the degradation of superstructure were caused by a lack of effective waterproofing path coverage leakage connection joints devices and a poor execution of concrete works (concrete low quality, insufficient, vibration inadequate formwork and support).

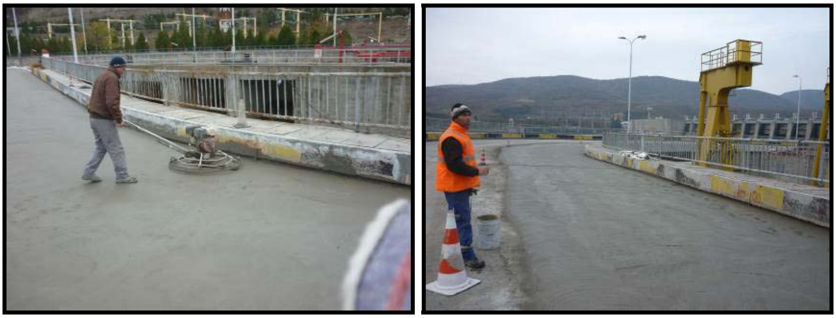

Fig. 13 – Execution of support concrete layer waterproof

Lack of effective waterproofing and sealing of the joints leading to water infiltration through path elements or joints and advanced degradation of concrete by corrosion.

A problem in the repair of the bridge carriageway consisted of uniformity of the bridge slab. Flatness was achieved by executing a leveling layer of concrete which have been created transversal and longitudinal slopes necessary for water drainage.

Fig. 14 – Waterproofing execution

Waterproofing of the bridge was carried out by using an advanced system.

The road bridge was completed by executing a layer of asphalt concrete and durable joint coverage devices.

Fig. 15 – Asphalt concrete and joint covering devices

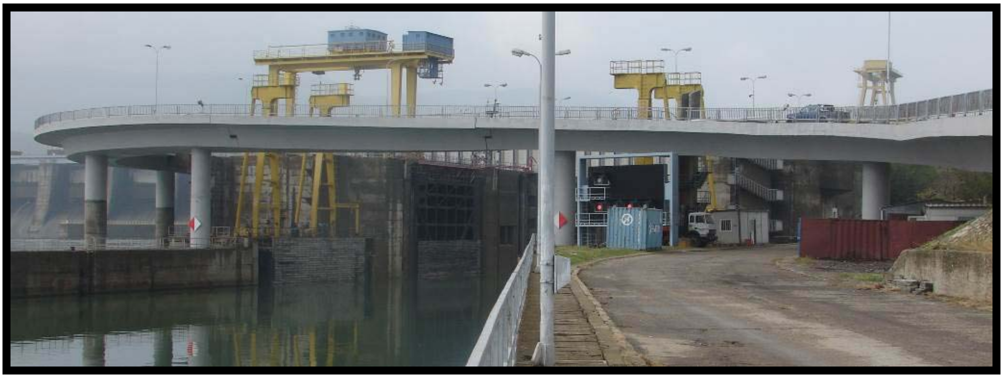

Fig. 16 – General view of bridge

3. CONCLUSIONS

Rehabilitation of the bridge has made with new materials with high resistance. Execution of works of such modern equipment needs, to be able to perform operations in conditions of maximum safety.

Acknowledgements

The authors wish to thank all those involved in project development and implementation.

References

1. Eurocode 1: Actions on structures. Part 2: Actions of traffic loads on bridges, European Committee for Standardisation, 2004
2. Eurocode 2 - Design of concrete structures. Part 2: Concrete bridges. Design and detailing rules, European Committee for Standardisation, 2006
3. STAS 1545-89, Road and street bridges. Loads, Romanian national design code
4. STAS 3221-86, Road bridges. Type trains and charging classes, Romanian national design code
5. G, Boaca, C.C. Comisu. " Rehabilitation of access bridge over the lock head downstream from the Iron Gates I SHEN", Contract 2/02.02.2010
6. Pardi L., Thogersen F., “Smart Structures: An European research funded project”, Proc. IABMAS, 2002.

Finite element method for arch bridge calculation

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Summary

This article presents an example for calculation of an arch bridge by analytical method for the calculation of static building and through computer programs.

In determining dead loads calculation efforts sectional arc method was achieved by transferring elastic unknowns in the center by creating a spatial model.

The effect of moving charges is determined using influence lines. To simplify the calculation using transmission lines direct influence hypothesis mobile loads. Calculation effort is made to specific sections of the arc (birth, quarter key).

In the computer program, efforts sectional were determined by repeating the calculation for positions convoy to 0.5 m

KEYWORDS: arch, elastic center, influence lines, finite element.

1. INTRODUCTION

Using specialized software for structural has acquired a significant increase during the last period. The development of strong computer allowed to create software with highly developed computer algorithms in the back which makes it easy for engineers.

Modern computing systems allow structural modeling and analysis of various constructive solutions. On models can be simulated results reveal future behavior of structures. To a large extent, the current computer aided design programs benefit analysis algorithms based on the finite element method. The advantage of this method is given by the flexibility and generality; it can be used in practically any field, from the calculation of bridge structures to electromagnetic analysis or study of fluids.

The main disadvantage of the finite element method is the fact that the accuracy of the results depends to a great extent on the experience and training of performing the analysis. This is not found in other calculation methods (such as eg boundary element method), so, unlike other methods, using finite element method requires a certain expertise and experience.

2. THE ARCH BRIDGE

The bridge has a static structure consisting of 2 unlimited elastic concrete arches, double recessed, way up in collaboration with elastic reinforced concrete deck continuous frames, columns and 2 longitudinal beams cross solidarity with cross pieces every 5.00 m and plate concrete on top.

Reinforced concrete arches has a rectangular section of constant width and variable height 0.60 m, 1.40 m and 1.00 m-key birthday. Arcs have a length of 40.00 m measured at birth line, providing a ratio f / l = 1/5, and were born in Piatra Neamt collected 0.40 m above in relation to the birth of the collected Bicaz. In cross section arcs are located at 5.00 m spacing, symmetrical directly related to the longitudinal axis of the bridge.

Arcs are secured to each other by diaphragms reinforced concrete rectangular section 0.60 x 0.30 m and 1.00 x 0.30 m field at recess, on the arcs. The link between arcs is provided with 4 diaphragms placed at 5.00 m from each other, along the bridge.

In the key, a length of 15.00 m arcs cast firmly attached to the deck beams, arches solidarity between deck spacers are provided, and the top plate reinforced concrete deck.

The resistance of the elastic concrete deck is made up of continuous reinforced concrete frames with rectangular pillars 0.50 x 0.60 m and longitudinal beams of rectangular section 0.35 x 0.75 m, with reinforced concrete slab variable height at the top. Cooperation between main beams is ensured by cross pieces of rectangular section 0.35 x 0.75 m located along the bridge to inter axle distance 5.00 m.

3. ARCH CALCULATION

Bridge structure calculation was performed in two ways: analytically, by methods of static constructions and computer programs. Analytical calculation was conducted tabular.

2.1. Analytical method

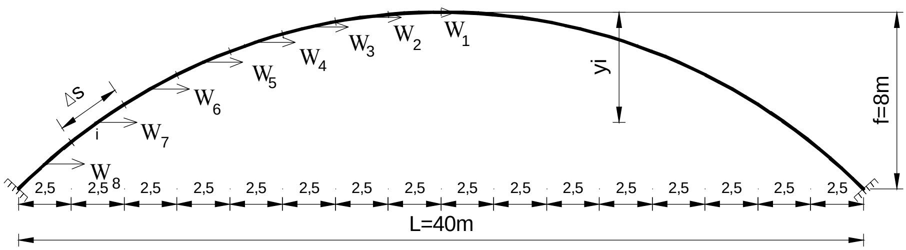
Analytical method of calculating arc followed these steps::

1. Establishing the basis system

2. Determination of basic equations

3. Determination of elastic center coordinates

Determinarea coordonatelor centrului elastic												
bolțar	secțiune	Δs (m)	b (m)	h_{med} (m)	A (m^2)	I (m^4)	I_0/I	W_i (m)	y' (m)	W_i*y' (m^2)	y=c-y (m)'
	0										
1	1	2,5	0,6	1,005	0,603	0,050	1,000	2,500	0,02	0,068	2,02
2	2	2,52	0,6	1,015	0,609	0,052	0,970	2,446	0,24	0,597	1,80
3	3	2,56	0,6	1,035	0,621	0,055	0,915	2,344	0,68	1,594	1,37
4	4	2,63	0,6	1,07	0,642	0,061	0,828	2,179	1,36	2,964	0,69
5	5	2,72	0,6	1,115	0,669	0,069	0,732	1,992	2,27	4,521	-0,22
6	6	2,85	0,6	1,17	0,702	0,080	0,633	1,806	3,47	6,268	-1,42
7	7	3,03	0,6	1,245	0,747	0,096	0,526	1,594	4,99	7,953	-2,94
8	8	3,25	0,6	1,345	0,807	0,121	0,417	1,356	6,88	9,328	-4,83
								Σ=16,217		Σ=33,292	



4. Evaluation of loads of weight concrete block

section	h (m)	b (m)	Δs (m)	γ_b (daN/m^3)	G (daN)
0	1,00	0,60		2500	
1	1,01	0,60	2,50	2500	3768,75
2	1,02	0,60	2,52	2500	3836,70
3	1,05	0,60	2,56	2500	3974,40
4	1,09	0,60	2,63	2500	4221,15
5	1,14	0,60	2,72	2500	4549,20
6	1,20	0,60	2,85	2500	5001,75
7	1,29	0,60	3,03	2500	5658,53
8	1,40	0,60	3,25	2500	6556,88

5. Assessment the permanent loads - Load Class E

Nr. Crt	Item name	g^n (daN/m^2)	n	g (daN/m^2)
1	Plate + girder (end)1,72x2500	4300	1,1	4730
2	Plate + girder (central)1,90x2500	4750	1,1	5225
3	(0,02x4,96)x2400	238,08	1,5	357,12
4	Waterproofing (0,01x4,96)x1500	74,4	1,5	111,6
5	Protection screed (0,04x4,96)x2500	496	1,5	744
6	Asphalt (0,06x3,96)x2400	570,24	1,5	855,36
7	Border (0,25x0,2)x2400	120	1,5	180
8	Filling sidewalk ((0,12+0,22)/2x0,8)x2400)	326,4	1,5	489,6
9	Pedestrian guardrail	90	1,5	135
10	Guide	90	1,5	135
11	Transversal beam (0,35x0,50)x2500	437,5	1,1	481,25

6. Displacement calculation δ11 , δ22, δ33

block	y*Wy (m^3)	x*Wx (m^3)	I_0/A (m^2)	I_0/Acosφ∆x (m^3)	∆s (m)	φ	cosφ	∆x (m)
1	10,231	3,906	0,084	0,210	2,5	1,93	0,999	2,498
2	7,979	34,400	0,083	0,207	2,52	6,82	0,992	2,500
3	4,399	91,554	0,082	0,200	2,56	11,89	0,978	2,504
4	1,038	167,229	0,079	0,190	2,63	16,95	0,956	2,514
5	0,096	252,534	0,076	0,177	2,72	22,13	0,926	2,519
6	3,642	341,997	0,072	0,162	2,85	27,6	0,886	2,525
7	13,776	421,898	0,068	0,143	3,03	33,42	0,834	2,527
8	31,631	477,178	0,063	0,121	3,25	39,61	0,77	2,503

7. Displacement calculation ∆1p, ∆2p, ∆3p, E load class

bolțar	Mp (daNm)	M^0 (daNm)	M^0*Wy (daNm^3)	M^0*W (daNm^2)	M^0*Wx (daNm^3)
1	27200,88	13600,44	-68784,213	34001,094	42501,367
2	105856	66528,44	-293917,652	162745,101	610294,127
3	238912,9	172384,4	-553523,068	404031,436	2525196,476
4	402973,3	320943,1	-482591,564	699408,064	6126814,642
5	679830,8	541402,1	237238,448	1078356,581	12142295,101

6	968627	824228,9	2114081,111	1488789,515	20485743,723
7	1377842	1173235	5497496,001	1869896,599	30423217,667
8	1814265	1596054	10452237,692	2164024,367	40597097,124
			16902236,756	7901252,756	112953160,228

8. Calculation of bending moments of dead loads –E class

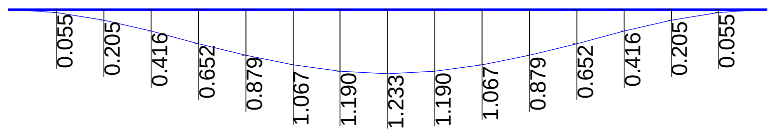
rost	M^i p	M_1^i	M_2^i	M_3^i	X_1	X_2	X_3	M_g^i (daNm)
0	0	2,05	0	-1	227786,8328	0	487220,97	-20258
1	27200,88	1,95	2,5	-1				-15835,8
2	105856	1,62	5	-1				-12350,3
3	238912,9	1,07	7,5	-1				-4576,18
4	402973,3	0,27	10	-1				-22745,2
5	679830,8	-0,79	12,5	-1				12658,24
6	968627	-2,13	15	-1				-3779,92
7	1377842	-3,8	17,5	-1				25031,1
8	1814265	-5,95	20	-1				-28287,6

9. Calculation of dead loads axial force

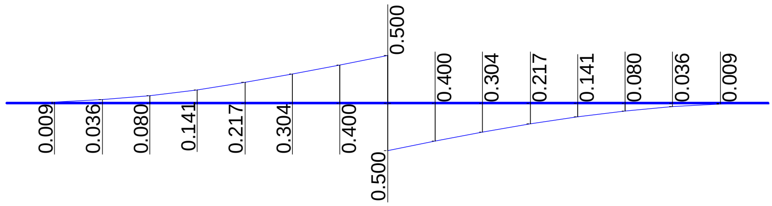
rost	φ	$\sin\varphi_i$	$\cos\varphi_i$	X_1	N^i_p	Ng^i (daN)
0	0	0	1	-227786,83	0	-227786,83
1	4,19	0,073064	0,997327	-227786,83	-1286,36	-228464,37
2	9,26	0,160915	0,986968	-227786,83	-5666,08	-230484,47
3	14,25	0,246153	0,969231	-227786,83	-12711	-233489,04
4	19,45	0,332984	0,942932	-227786,83	-29030,8	-243818,36
5	24,74	0,418501	0,908216	-227786,83	-38390,3	-245270,01
6	30,41	0,506184	0,862425	-227786,83	-66324,6	-262774,73
7	36,42	0,5937	0,804687	-227786,83	-81151,1	-264448,11
8	42,9	0,680721	0,732543	-227786,83	-110665	-277528,72

10. Determination of influence lines for moving loads

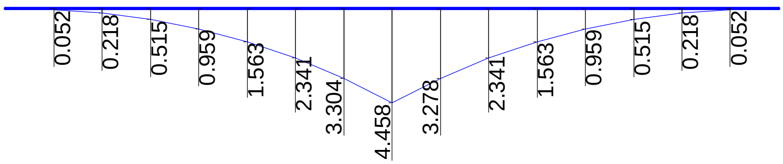
Effect of mobile loads is determined using influence lines. Since the unknowns X1, X2, X3, in the elastic center is determined from independent equations, the most common method for determining lines of influence, is to first determine influence lines of unknowns X1, X2, X3, which then deduced by simple overlapping effects influence line of any effort from any section.



Influence line when unknown X_1=1

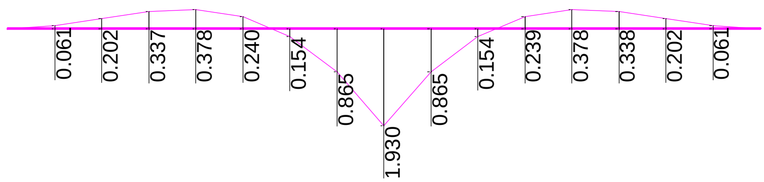


Influence line when unknown X_2 =1

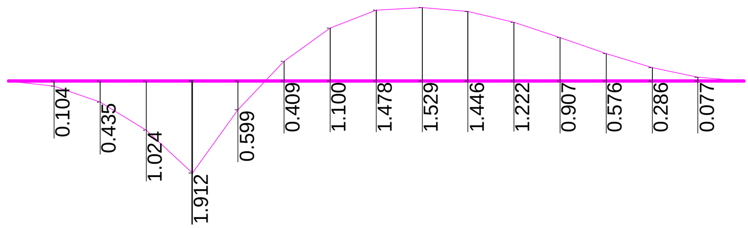


Influence line when unknown X_3 =1

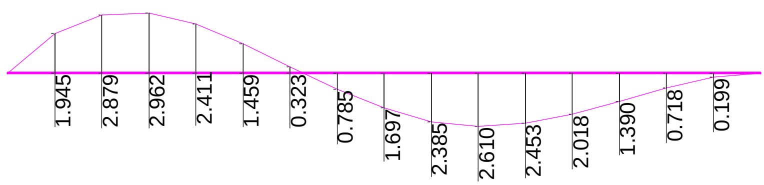
11. Lines of influence of efforts sectional



Influence line for the moment in the key

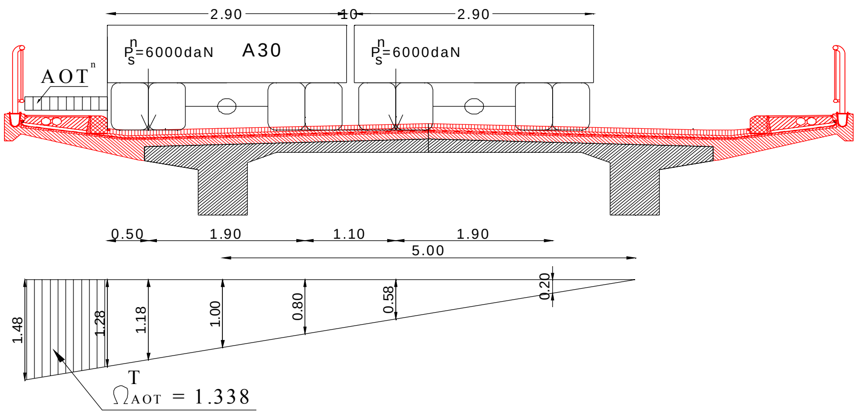


Influence line for the moment in the quarter

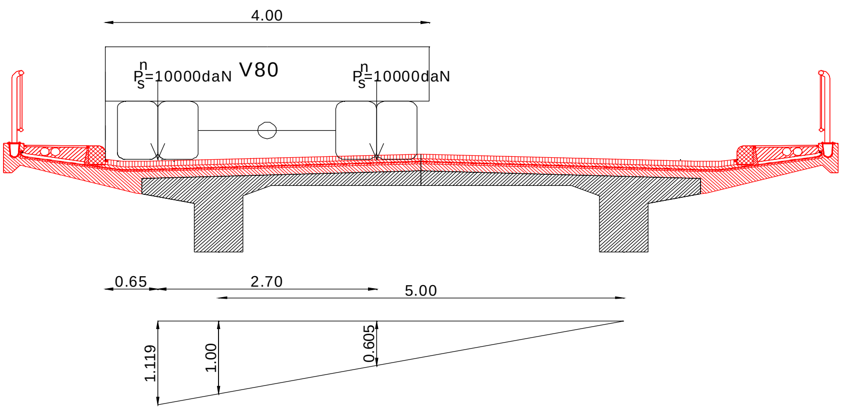


Influence line for the moment in the birth

12. Position calculation convoys in transverse direction

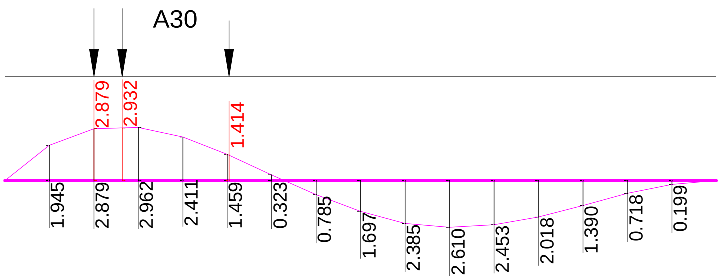


Determination of efforts in the cross sectional from A30 and AOT

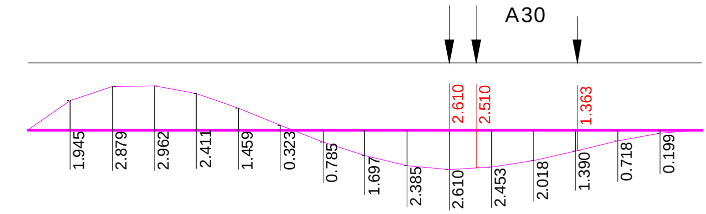


Determination of efforts in the cross sectional for V80

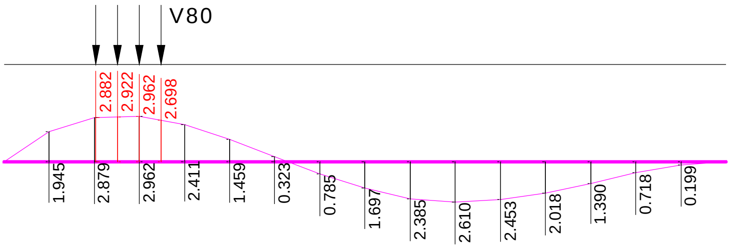
13. Position of calculation convoys on the line of influence



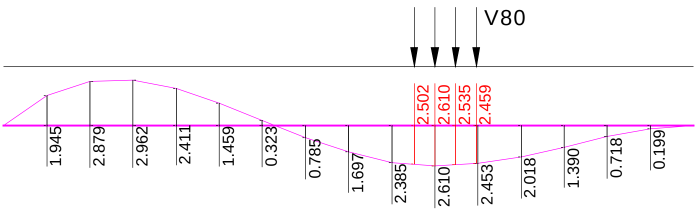
Influence line for the moment, minimum in birth A30



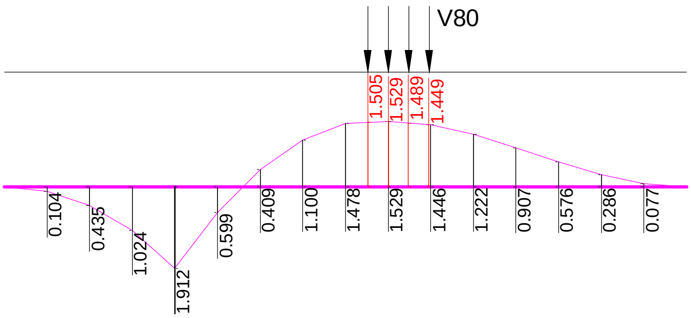
Influence line for the moment, maximum in birth A30



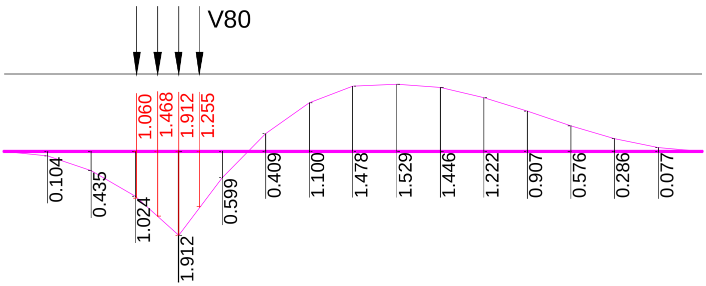
Influence line for the moment, minimum in birth A30V80



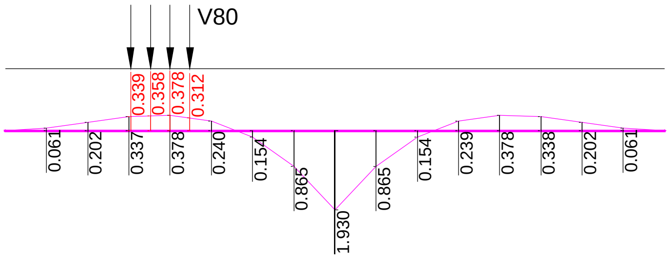
Influence line for the moment, maximum in birth V80



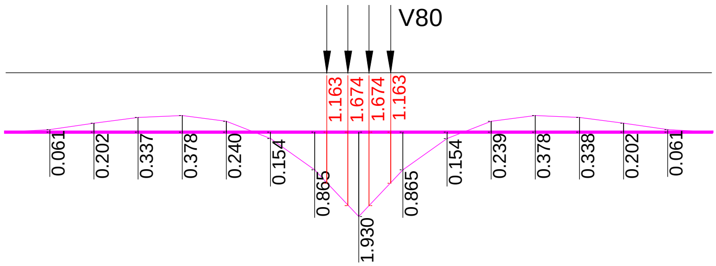
Influence line for the moment, minimum in quarter V80



Influence line for the moment, maximum in quarter V80



Influence line for the moment, minimum in key V80



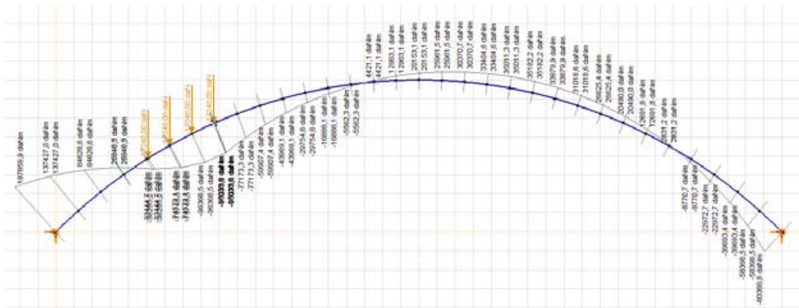
Influence line for the moment, maximum in key V80

14. Centralization of bending moments

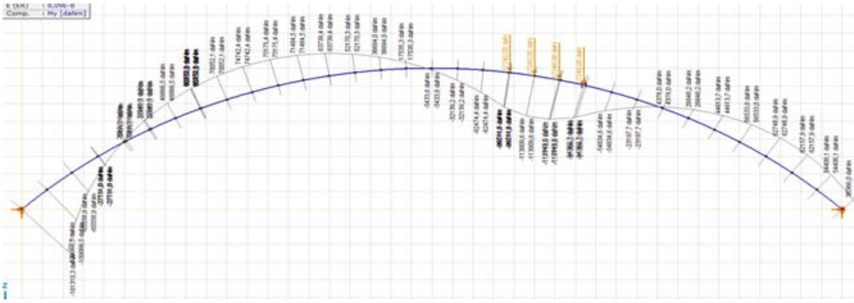
Effort	Section		Class E
M (daNm)	Birth	Maxim	208796,88
		Minim	237236,4
	Quarter	Maxim	100762.16
		Minim	141535.69
	Key	Maxim	8415.56
		Minim	137434.83

2.2. 2D – Arch calculation

In order to verify the correctness of calculations, carry out a check for the same static diagram in a computer program.



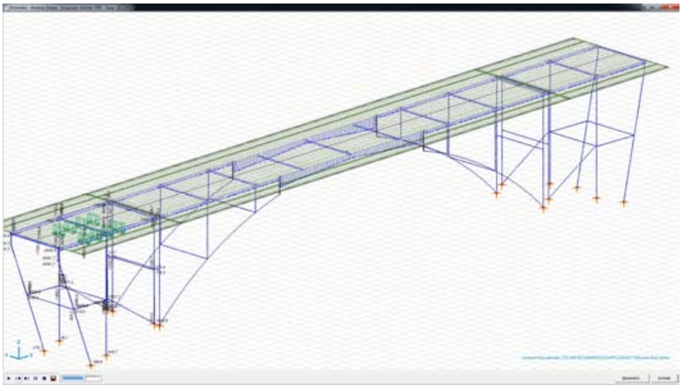
Arch – Maximum bending moment from convoy V80



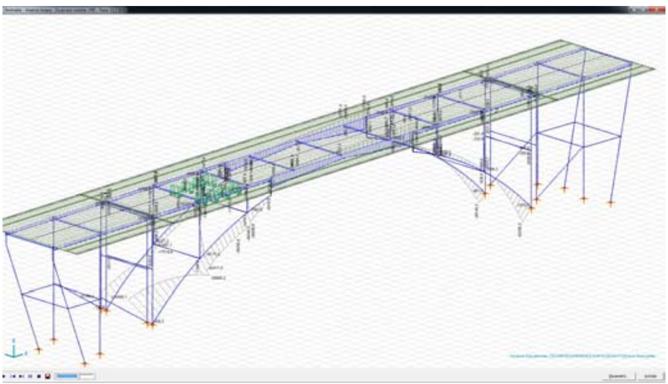
Arc – Minimum bending moment from convoy V80

2.3. 3D – Arch calculation

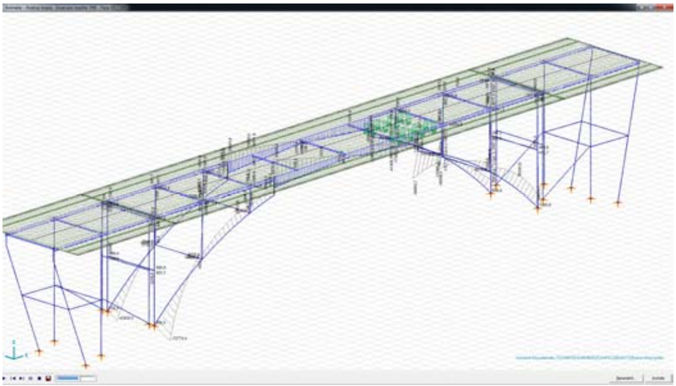
In the last variant has been created the 3D model of the bridge and loaded with a mobile charging. Using computer programs allow you to determine quickly stored over tensions and twisted bridge structure.



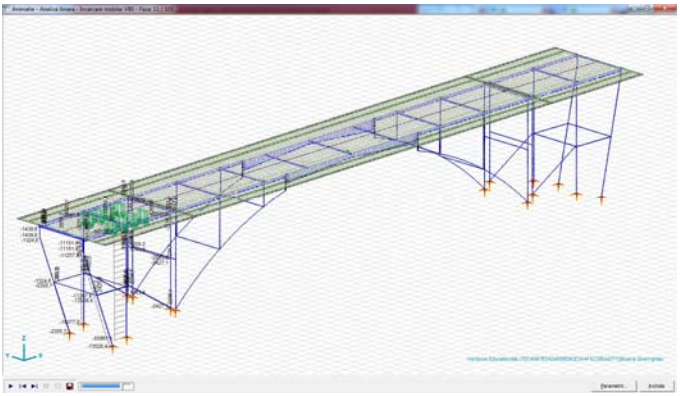
Convoy V80 on abutment



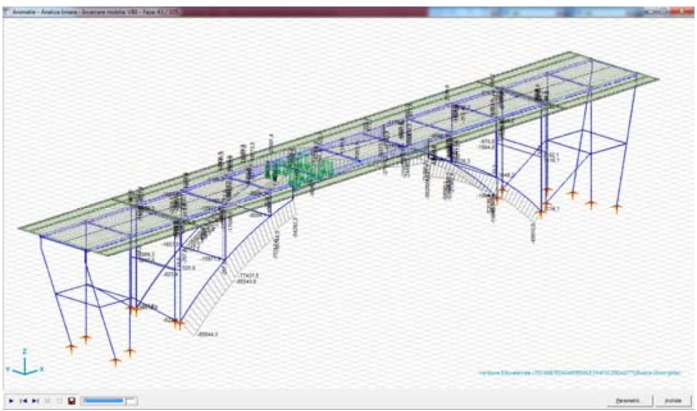
Maximum bending moment in the birth from the convoy V80



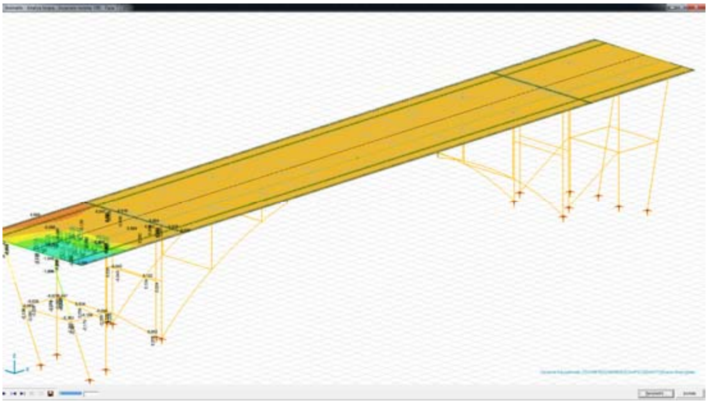
Minimum bending moment in the birth from the convoy V80



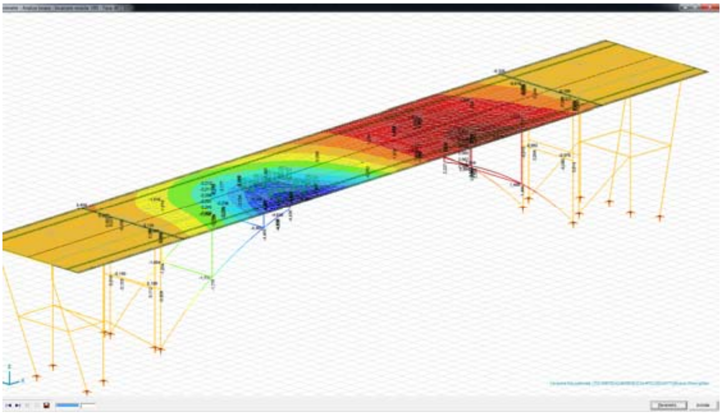
Axial force from V80 – Abutment



Axial force from V80 – Maxim on half



Abutment – Deformations diagram from convoy V80



Arch – V80 convoy deformations diagram from the third

3. CONCLUSIONS

Manual calculation, coupled with computer using software brings extra safety. Exclusive use of programs without making periodic checks can lead to errors in model creation and / or the application of loads.

References

1. C.C.Comisu , Poduri de beton armat, Editura Societății Academice "Matei Teiu Botez"
2. P. I. Radu – Poduri din beton armat, Editura tehnică Bucuresti 1983
3. P. Mazilu – Statica construcțiilor vol.II Editura tehnică Bucuresti 1959
4. C. I. Amariei Statica construcțiilor vol. II Iasi 1981Achkire, Y., Preumont, A., Active tendon control of cable-stayed bridges, *Earthquake Engineering and Structural Dynamics*, vol. 25, 1996.

Trans-Risk-Analist, a decisional model, based on risk analysis and management, for the evaluation of road infrastructure projects

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Summary

There is work going on in many countries with regard to risk analysis and the need to improve the decision making process regarding the management of infrastructural projects. The purpose of this work is to take better decisions and improve the accuracy regarding construction costs and time estimation.

This paper presents a decisional model, based on risk analysis and management, for the evaluation of road infrastructure projects. Risk analysis gives the possibility of differentiating the feature of risk information in terms of outcome criteria such as net present value, the internal rate of return or the benefit/cost rate. This differentiation is made by using probability distributions. The risk model proposed is structured so that it can be easy to use, flexible and functional. The main advantage in using this model is that it helps the decision makers in taking better informed decisions.

KEYWORDS: risk management, infrastructure, road transport.

1. INTRODUCTION

General tendency of underestimation of costs investments and overestimation of benefits (demand forecast/prognosis) reveals that socio-economic analysis become over-optimistic leading to wrongful decision support.

As an example the statistical data shows that in almost 9 out of 10 projects costs are underestimated. For a randomly selected project, the likelihood of actual costs being larger than estimated costs is 86%. [1]

A recent study shows that, for road projects, in 50% of the cases considered the difference between actual and forecasted traffic is more than 20%, as presented in Figure 1.

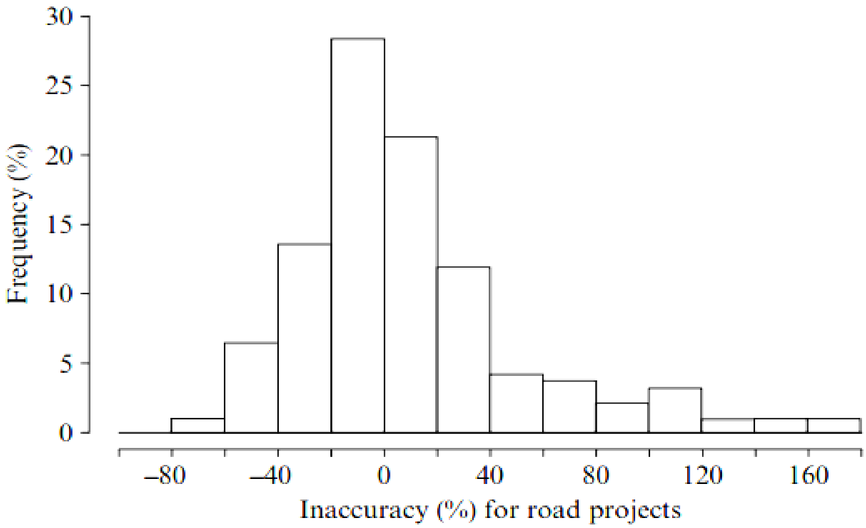

Figure 1. Inaccuracies of travel demand forecasts in transportation infrastructures projects on 183 road projects. [1]

Based on a study financed by the European Union there are presented a list of impacts that are subjected to risk, for the road infrastructure projects.[2]

Core impacts

- A1 Investment
- A1 Investment costs
- A2 System operating and maintenance costs
- A3 Vehicle operating costs
- A4 Travel time benefits Car
- A5 Safety Casualty related costs
- A6 Local environment Noise

Non-core, non-strategic impacts

- B1 Service Quality
- B2 Landscape Indication of severe, moderate, slight visual intrusion

Strategic, territorial impacts

- C1 Strategic mobility (accessibility)
- C2 Strategic environment Global air pollution (CO_2),
- C3 Strategic economic development

Strategic, non-territorial impacts

- D1 Private financial attractiveness
- D2 Other strategic policy and planning

Having as a starting point the study mentioned above but also other similar studies and reports, the risk analysis model proposed is taking into consideration and incorporate in his structure those impacts mentioned above.

The main focus of this paper is on the structure of the proposed model and what data is needed in order to reach trustful results, in order to diminish the level of inaccuracies. The software used in the proposed model formulation is based upon a Microsoft Excel platform with add-on software implementing the risk analysis named @RISK from Palisade.[3]

2. MODEL STRUCTURE

The model, which has the generic name Trans-Risk-Analist, has the structure presented in Figure 2.

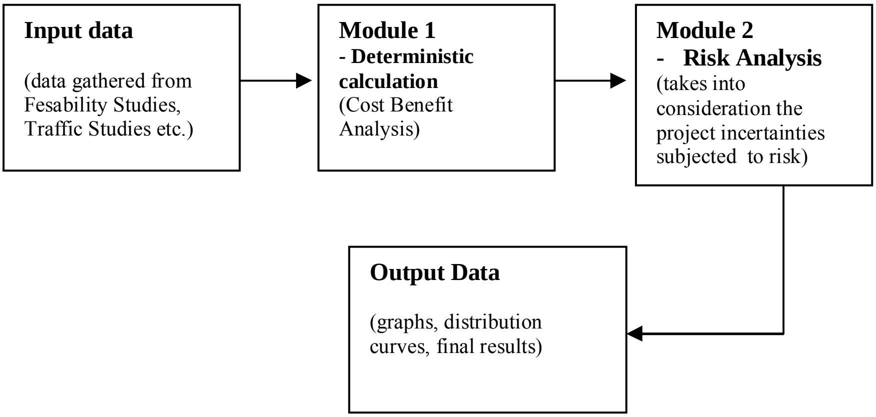

Figure 2. Trans-Risk-Analist internal structure.

The deterministic calculation consists of a couple of worksheets set out as a top-down approach. The entry or input sheet, as shown in Figure 3, currently consists of a couple of possible input categories subject to change. Additional entries are construction costs (investment costs), sequentially divided operating and maintenance costs, evaluation period and other key parameters. The ‘yellow’ entry fields denote user input. These numbers are possible to change, but as default they are pre-determined.

To deal with this the risk analysis together with other simulation (i.e. Monte Carlo Simulation) based on reference class forecasting is applied for determining the output distribution for benefit cost ratio instead of conventional single point estimate. This is presented by the certainty values and graphs or probability distributions.

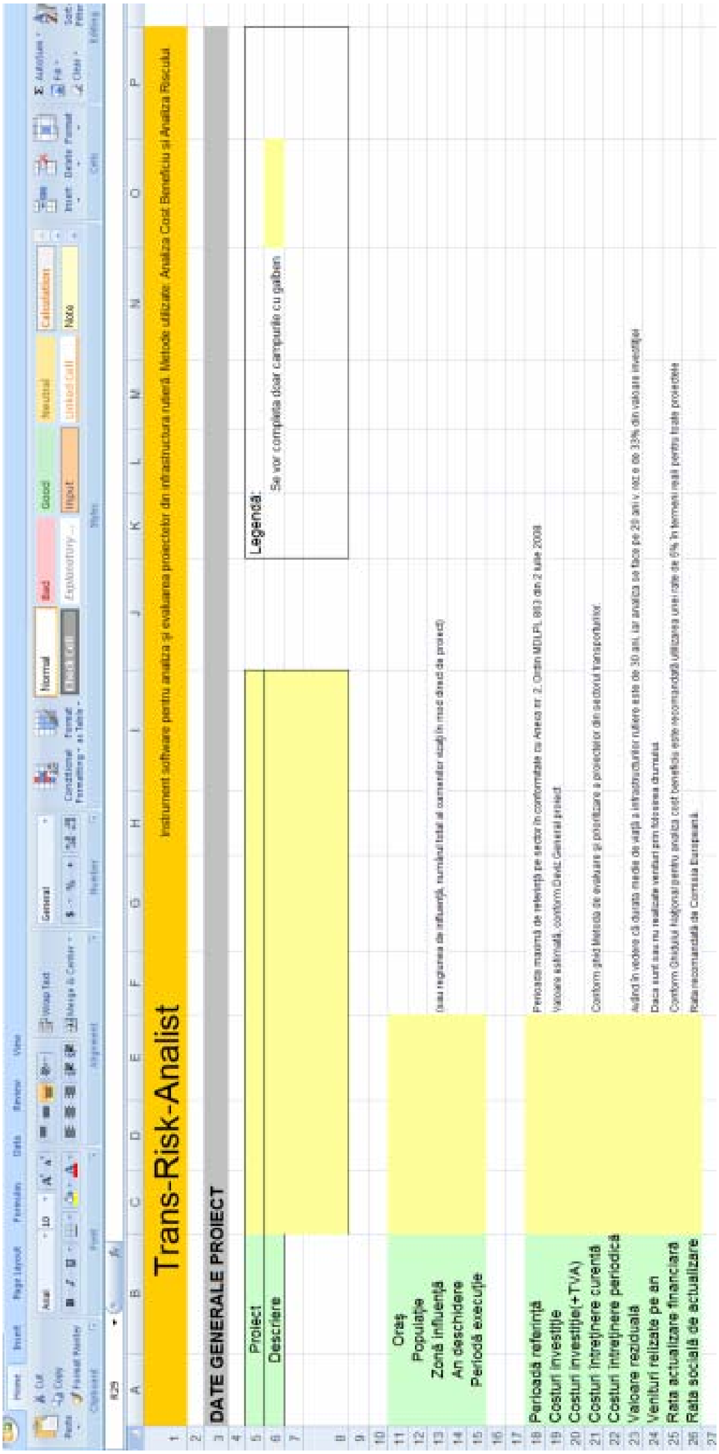

Figure 3. Dumb screen from Trnas-Risk-Analist, input data sheet.

In order to elaborate a realist analysis it is compulsory to take into account some estimations regarding the cost and benefits of the project and some working variables which will be used as starting point variables, Figure 4.

The results from the deterministic module are presented in different tables which show the values for net present value, internal rate of return and cost benefit ratio.

Trans-Risk-Analist

Instrument software pentru analiza şi evaluarea proiectelor din infrastructura rutieră. Metode utilizate: Analiza Cost Beneficiu, Analiza Riscului şi Analiza Multi Criterială.

Variabile de lucru	Ipoteze
Orizontul de analiză	Orizontul de analiză este de 23 ani, între 2010 ÷ 2032, din care trei ani de execuţie a lucrărilor, între 2010 ÷ 2012.
Costurile totale ale proiectului	Sunt luate în considerare costurile investiţiei şi cele de întreţinere şi operare.
TVA	În devizul general al investiţiei s-a considerat TVA 24% Aceasta a fost eliminată din valoarea investiţiei la elaborarea analizei cost beneficiu.
Valoarea reziduală	Având în vedere că durata medie de viaţă a infrastructurilor rutiere este de 30 ani iar analiza se face pentru 20 ani de operare, valoarea reziduală la finele orizontului de timp este de 33% din valoarea investiţiei.
Rata de actualizare financiară	5% - rata recomandată de Comisia Europeană.
Rata socială de actualizare	5,5% - rata recomandată de Comisia Europeană pentru "ţările de coeziune", deci şi pentru România.

Sector	Perioada de referinţă (ani)
Energie	15 ÷ 25
Apă şi mediu	30
Căi ferate	30
Porturi şi aeroporturi	30
Drumuri	25 ÷ 30
Industrie	10
Alte servicii	15

INDICATORI DE PERFORMANŢĂ ECONOMICĂ	
VALOAREA NETĂ PREZENTĂ ECONOMICĂ (mii €)	54.046,10
RATA INTERNĂ A RENTABILITĂŢII ECONOMICE (%)	16,96%
RAPORT beneficii actualizate/ costuri actualizate	2,98

Figure 4. Dumb screen from Trans-Risk-Analist, hypothesis, working variables and results from deterministic module.

Simulation models, such as the model presented in this thesis, use random variables as input stated as randomized probability distributions for which reason the simulation output data themselves are random. [4] Care must be taken in drawing conclusions about the model's true characteristics both concerning the random variables and the involved correlations. The chosen impacts used for the MCS are all assumed uncorrelated, hence no interdependencies are present. The actual Monte Carlo simulation shown in Figure 5 is based upon the two sets of previously mentioned parameters and distributions, rooted in the epistemic and ontological uncertainties.

The purpose of the Trans-Risk-Analist result sheet is to provide the decision-makers with a mean for widen their assessment of the possible BCR. Specifically, Figure 5 shows three reports based on @RISK: Histogram showing the most frequent BCR, a descending accumulated graph that shows the "certainty" of achieving a certain BCR or better and finally a correlation tornado graph that illustrates the impact (correlation) of each variable or parameter to the overall BCR. Obtaining a probabilistic view of the BCR is especially beneficial when several projects are to be evaluated. The possibility of applying, e.g. different scenarios, evidently by various input parameters creates varying degrees of uncertainty expressed by the steepness of the descending accumulated graph. [5]

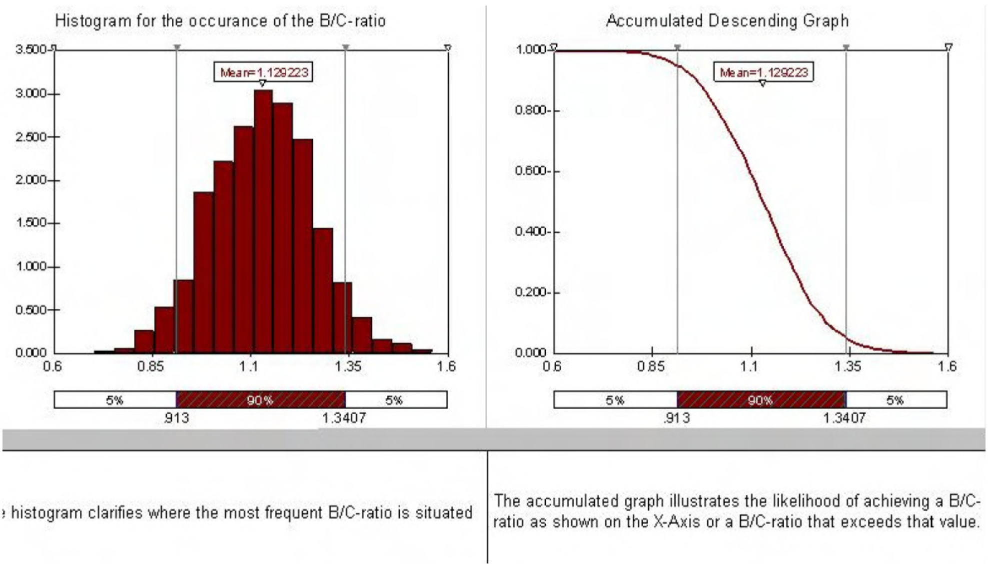

Figure 5. Monte Carlo simulation diagrams and graphs obtained by using @Risk.

At the end, after passing through all the steps, the program displays an overview of the most important results in a sheet alike the one presented in the Figure 6.

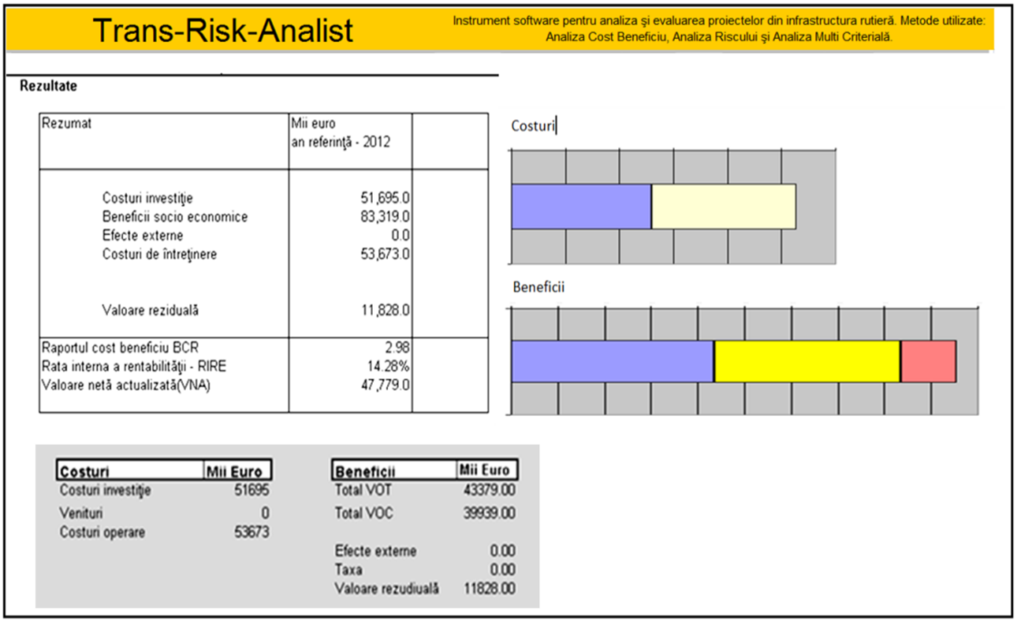

Figure 6. Dumb screen, final result sheet from Trans-Risk-Analist

3. CONCLUSIONS AND PERSPECTIVES

With Trans-Risk-Analyst it is possible to carry out a project appraisal study according to the principles described above but also by adding other criteria. The software model has been designed as a combined approach in determining the feasibility of a road infrastructure project by use of both a deterministic and a stochastic approach based on @RISK. Thus a deterministic point estimate and a stochastic interval measure make it possible to assist the decision-makers by an accumulated graph whereby risk aversion can be taken into consideration.

The decision support model will be further developed in future case studies and in the doctoral thesis "Risk management for road transport infrastructure".

Acknowledgements

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References

1. Bent, F., Niels, B., Werner, R., *Megaprojects and Risk: An Anatomy of Ambition*, Cambridge University Press, United Kingdom, 2003.
2. Eunet/SASI Final Report, Socio Economic and Spatial Impacts of Transport, Project Funded by the Europea Commission under the transport RTD Programme of the 4^{th} Framework Programme, 2001.
3. Palisade Corporation, @RISK – Risk Analysis and Simulation Add-In for Microsoft Excel. Version 5.0, *User Manual*, Newfield, USA, 2007.
4. Vose, D., *Risk Analysis – A Quantitative Guide,* 2nd Edition, John Wiley and Sons Ltd., England, 2000.
5. Leleur, S., Holvad, T., Salling, K.B. and Jensen, A.V., *Development of the CLG-DSS Model. Midterm Report presenting the CTT contribution to TASK 9*, Centre for Logistics and Freight Transportation, Denmark, 2004.

Road network design method applied to the Romanian road network

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Summary

The Romanian road network consists mainly of national roads, and county roads. The network also has few kilometers of motorways, comparing to the surface of the country, and a few motorway sectors which are under construction or are planned to be constructed in the near future. The last traffic counts show us that this road network can not deal with the actual traffic load, not to speak about the future loads. So it is imperatively necessary to invest in building motorways with proper capacity, roads that should connect the most important cities(centroids) in the country. These centroids will further be classified and chosen.

A simple methodology of creating a road network, has been developed by Immers et al.(2010)[1]. The methodology has been applied in other countries (Hungary, state of Florida - USA), and it was demonstrated that it is an efficient approach. This methodology can also be applied to Romania as well, considering the fact that the motorway network may be considered almost nonexistent. Therefore, proposals about motorway extensions will be made in this article.

KEYWORDS: centroids, design methodology, motorway network, traffic load.

1. INTRODUCTION

A number of **16.062 km** of national roads are being used at this moment. A total number of **1359 km** of motorways will be in operation in the near future, from which **519 km** are already in use.

A simple methodology of creating a robust road network, has been developed by Immers et al.(2010). This methodology can also be applied to Romania as well, considering the fact that the motorway network may be considered almost nonexistent. Therefore, proposals about motorway extensions will be made in this article

The first step of the methodology is to classify the centroids and to establish those ones who will be integrated in the motorway network.

2. STEP1: HIERARCHY OF CENTROIDS

A classification of centroids will be made according to the size of them, and based on the number of inhabitants. This information is taken from the last census measurements, which are not final and official yet. But an overview about the real situation can be made using the partial results of the census.

Romania has a population of approximately 19 million inhabitants and lies on a surface of 238.000 square km, resulting a density of 80 inhabitants/square km.

The centroids will be classified in 3 categories:

- Bucharest and the main abroad cities
- Centroids with population greater than 200.000 inhabitants
- Centroids with population greater than 70.000 inhabitants.

Bucharest and the main external centroids have to be considered the main economical directions, so these should be the most important connections that must be incorporated. The international connections are the most important links, and Romanian motorways must be part of the European road network. On Romanian territory should be two main European corridors of motorways, named the IVth and the IXth European corridor. The route of the IVth European corridor is: *Dresde/Nurenberg - Prague - Vienna - Bratislava - Gyor - Budapest - Arad - Bucharest - Constanta/Craiova - Sofia - Thessaoloniki/Plovdiv - Istanbul [2].* On Romanian territory, this corridor will follow the next path: *Nadlac - Arad - Timisoara - Lugoj - Deva - Orastie - Sibiu - Pitesti - Bucharest - Constanta.* The route of the IXth European corridor is: *Helsinki - Vyborg - St. Petersburg - Pskov - Gomel - Kiev - Ljubashevka - Chişinău - Bucharest - Dimitrovgrad - Alexandroupolis [2]*. This motorway corridor is planned to cross Romania through: *Albita - Focsani - Buzau - Ploiesti-Bucharest-Giurgiu*

Connections with other European major cities must be also included, because Romania is part of European community. Hence, connections should be made with the eastern part of Europe, especially with the biggest cities.

The international connections, will be considered the cities with a population of about 1 million inhabitants, more or less, and that are located at distances nor exceeding about 600 kilometers of the border.

1. Sofia (1.204.685 inhabitants)
2. Istanbul (13.483.052 inhabitants)
3. Belgrade (1.154.589 inhabitants)
4. Budapest (1.737.000 inhabitants)
5. Lvov (756.618 inhabitants)
6. Chisinau (723.500 inhabitants)
7. Kiev (2.797.553)
8. Odessa (1.003.705 Inhabitants)

There will be considerated a number of 28 national cities, the most important of them being the capital Bucharest, city which has a population of about 1.7 million inhabitants. As mentioned above, the main criteria to choose the centroids will be the number of inhabitants. The cities that were chosen for designing the motorway national network are:

1. Bucharest (1.677.985 inhabitants)
2. Cluj-Napoca (309.136 inhabitants)
3. Timisoara (303.708 inhabitants)
4. Iasi (263.410 inhabitants)
5. Constanta (254.693 inhabitants)
6. Craiova (243.675 inhabitants)
7. Galati (231.204 inhabitants)
8. Brasov (227.961 inhabitants)
9. Ploiesti (197.552 inhabitants)
10. Oradea (183.123 inhabitants)
11. Braila (168.389 inhabitants)
12. Pitesti (148.264 inhabitants)
13. Arad (147.922 inhabitants)
14. Sibiu (137.023 inhabitants)
15. Bacau (133.460 inhabitants)
16. Targu Mures (127.849 inhabitants)
17. Baia Mare (114.925 inhabitants)
18. Buzau (108.384 inhabitants)
19. Botosani (100.890 inhabitants)
20. Satu Mare (94.948 inhabitants)
21. Ramnicu-Valcea (92.573 inhabitants)
22. Drobet-Turnu-Severin (86.475 inhabitants)
23. Suceava (86.282 inhabitants)
24. Targu-Jiu (78.547 inhabitants)
25. Piatra-Neamt (77.393 inhabitants)
26. Targoviste (73.964 inhabitants)
27. Focsani (73.868 inhabitants)
28. Bistrita (70.493 inhabitants)[3]

These 28 centroids are the most important cities from Romania. Of course Bucharest stands on top of the other, this city being the 10th in an European city ranking.

All the 28 cities have a population of 5.814.069 inhabitants, which represents 30.53% from the total of 19.042.936 of inhabitants living in Romania.

These are some statistics and a big picture can be seen, regarding the ranking of Romanian cities, in our case centroids, emphasis will be on Bucharest, the capital, and the connections with other important cities from other parts of country.

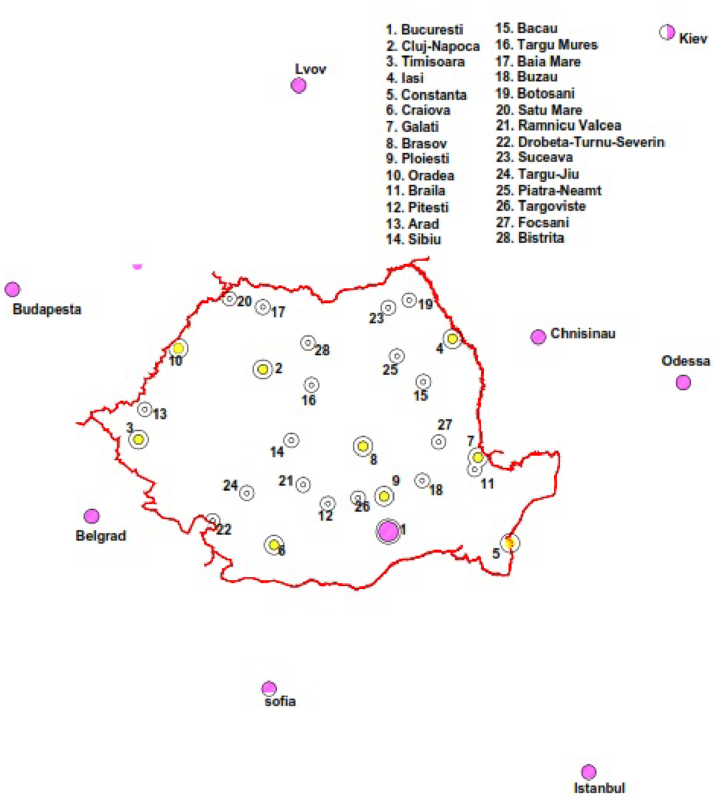

Figure 1. Main centroids

3. STEP2: INTERNATIONAL CONNECTIONS.

The second step in creating a motorway network would be establishing the international cities with whom Bucharest should be connected. Were considered also, the most important external centroids, with population of about 1 million inhabitants, and that are located at distances not exceeding about 600 kilometers of the border.

Bucharest has no direct connections through motorways with other important abroad cities. So this could be an advantage, meaning that, for this, Romanian authorities could make the best connections using the many researches that were made until now, and, why not, this methodology which will lead to finding the very important links and to a prioritization of investments.

The most important connections will be:

- Bucharest - Sofia(Bulgaria, Greece)

Sofia is the most important city from Bulgaria and the capital, with a population of 1,204,685 inhabitants [4]. From here, many other paths can be followed, for instance to Athens, to Belgrade, to Macedonia, Albania, Serbia. The distance from Bucharest to Sofia is about 300 km on the actual road network.

- Bucharest - Istanbul(Turkey)

The second international connection should be to Istanbul, the biggest city in Turkey, which lies on two continents, and has a population of 13,483,052 [5]. Even if the distance could be considered to big, Istanbul remains one of the most important destinations in this part of Europe. Connections between Romania and Turkey, specifically Istanbul, are well known from past, and it remained quite the same. More, speaking in terms of tourism, Turkey is one of the most visited countries in Europe, so a direct connection is welcome.

Of course, some might say that we can not make a direct link to Istanbul, because we have to pass Bulgaria, but we can assume that the Bulgarian authorities would use the same principle while they are building their own motorway network.

- Bucharest - Belgrade(Serbia)

The next international connection is with Belgrade, the capital of Serbia. This is the biggest city in this country, and from the ex Republic of Yugoslavia, with a population of 1,154,589 [6] inhabitants. The distance from Bucharest is around 600 km.

The economic relationships could be made between Romania and Serbia via Danube, Belgrade being one of the four capitals which are crossed by this river. Bucharest is not crossed by Danube, although distance till Danube would not be very big, but this transportation system is not well developed in Romania, and a direct connection is more convenient for both cities.

- Bucharest - Budapest (Hungary, Austria, Czech Republic)

The most important external connection is, for sure, with Budapest, the capital of Hungary. This city has a population of 1,737,000 [7] inhabitants and is the main "gate" to the western part of Europe. The distance to Budapest, from Bucharest is around 850 km, but a direct connection is now under construction, and it is estimated that all the 594 kilometers of A1 motorway (on Romanian territory) will be finished in 2016.

Also, another connection with Budapest will be made in the future through A3 motorway, which is under construction, and it is estimated that all the 584 kilometers will be finished until 2020.

From here, many other connections can be made, to other citieis from western European countries, so we may conclude that this is the most important external connection with Bucharest.

- Bucharest - Lvov(Ukraine, Poland)

Lvov is one the largest cities from the western part of Ukraine, with a population of 756,618 [8] inhabitants. It is not the most important city in Ukraine, but making a connection with this part of Europe is necessary, from this route it can be reached destinations from Slovakia and Poland. The distance from Bucharest to Lvov would be around 850 km. Even if a direct connection can be made, perhaps more convenient, through the north-eastern part of Romania, via Suceava (centroid 22), a connection in the north-west part of Romania is needed.

- Bucharest - Chisinau, Kiev(Moldavia, Ukraine)

Chisinau is the capital and the biggest city of Moldavia, with a population of 723,500 [9] inhabitants. Many historians consider that it is the second Romanian city after Bucharest, because Moldavia was part of Romania, so the relations between the two countries are very close. The distance from Bucharest to Chisinau is around 400 km. This link between Bucharest and Chisinau is basically part of the 4th European corridor.

Besides that, this connection would be the most important one to Kiev, the capital of Ukraine. This is the biggest city in Ukraine, with a population of 2,797,553 [10] inhabitants. From here connections to Russia, Belarus, and other Baltic countries can be made. The distance from Bucharest to Kiev via Chisinau would be approximately 950 km.

- Bucharest - Odessa(Ukraine)

The last international centroid which has to be considered is Odessa, a big Ukrainian city located on the Black Sea coast, with a population of 1,003,705 [11]. In this case as well, economical relations can be made on the river and sea, and for developing this relations, building a motorway connection is welcome. This city could not be considered as an important one, but it should be, because has a great population, is an important harbor at the Black Sea, tourism is growing fast in Ukraine, so it should be treated as an important abroad city.

- Bucharest - Constanta.

Of course Constanta is part of Romania, but is the greatest harbor from this country so a lot of trade is doing on there, so it will be considered as being an external node.

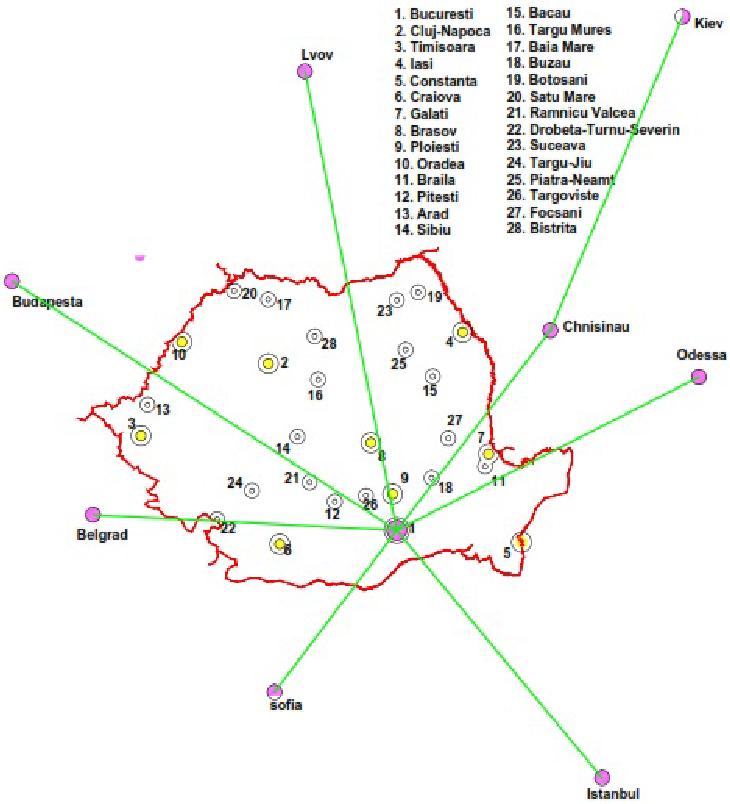

Figure 2 - International connection

4. STEP 3: DESIGN OF THE IDEAL NATIONAL NETWORK.

The next step of the methodology is to design a road network that is supposed to be perfect, considering, first of all, the connections with the external centroids, and trying to connect the main cities from Romania. This network should be done considering that there is no network, that all the centroids have no connections, and this road network is the first one to be built (from scratch)

The road network is based on the connections of Bucharest with the rest of the regions from Romania. The first 10 cities will be considered the most important ones (more than, or around 200.000 inhabitants), that are arranged radially on all directions of the country.

Connections with international cities will be made trying to connect as many as possible internal centroids, especially the second level ones, namely:

1. Bucharest (1.677.985 inhabitants)
2. Cluj-Napoca (309.136 inhabitants)
3. Timisoara (303.708 inhabitants)
4. Iasi (263.410 inhabitants)
5. Constanta (254.693 inhabitants)
6. Craiova (243.675 inhabitants)
7. Galati (231.204 inhabitants)
8. Brasov (227.961 inhabitants)
9. Ploiesti (197.552 inhabitants)
10. Oradea (183.123 inhabitants)

The third category of centroids will be connected afterwards, with lower level of motorways. These links will connect all centroids, without letting unconnected any city. The third category are the cities with a population greater than 70.000 inhabitants:

11. Braila (168.389 inhabitants)
12. Pitesti (148.264 inhabitants)
13. Arad (147.922 inhabitants)
14. Sibiu (137.023 inhabitants)
15. Bacau (133.460 inhabitants)
16. Targu Mures (127.849 inhabitants)
17. Baia Mare (114.925 inhabitants)
18. Buzau (108.384 inhabitants)
19. Botosani (100.890 inhabitants)
20. Satu Mare (94.948 inhabitants)
21. Ramnicu-Valcea (92.573 inhabitants)
22. Drobet-Turnu-Severin (86.475 inhabitants)
23. Suceava (86.282 inhabitants)
24. Targu-Jiu (78.547 inhabitants)
25. Piatra-Neamt (77.393 inhabitants)
26. Targoviste (73.964 inhabitants)
27. Focsani (73.868 inhabitants)
28. Bistrita (70.493 inhabitants)

The first international connection, Bucharest-Sofia, should be made directly, more or less, near Giurgiu, a place where is one of the most important custom with Bulgaria.

The second link, with Istanbul can be made also directly, without connecting any of the cities from the list above. Of course, these two links, to Sofia and Istanbul could merge on the Romanian territory, somewhere between the intersections proposed in this methodology, or on the same path as to Sofia, but this is a fact that should consider the Bulgarian view on how they will build their own motorway network. Also another possibility to get to Istanbul can be by traveling to Constanta(5) and then along the Black Sea shore. Hence, we will consider that the optimal path for traveling to Istanbul, with reference to the Romanian territory, will be the same link as for Sofia.

The next external centroid that should be connected to Bucharest is Belgrade, the capital of Serbia. Connection will be made like this:

- Bucharest(1) - Craiova(6) - Drobeta-Turnu-Severin(22) - Belgrade.

In this case there are not many options, this would be, largely, the optimal path for the international connection Bucharest-Belgrade.

The most important external city is Budapest. In order to connect the two large cities, Bucharest and the capital of Hungary, there are some motorway links, which are being used till now, but there are also some links under construction. There are few alternatives of some different paths, but the best one is to follow the IX-th European corridor route.

- Bucharest(1) - Pitesti(12) - Ramnicu Valcea(21) - Sibiu(14) - Timisoara(3) - Arad(13) - Budapest.

As alternative to this route, Drobeta-Turnu-Severin(22) should be connected with Timisoara(3) so it should be possible to go through this route to get to Budapest:

- Bucharest(1) - Craiova(6) - Drobeta-Turnu-Severin(22) - Timisoara(3)

Another way to get to the Hungarian motorway network will be the next one

- Bucharest(1) - Ploiesti(9) - Brasov(8) - Targu Mures(16) - Cluj-Napoca(2) - Oradea(10)

This is also an under construction motorway which crosses mountain areas and connects plenty of important cities from Romania.

For the connection with Lvov, there will be used the same links as for second connection to Budapest, only that from Cluja-Napoca(2) will switch to north.

- Bucharest(1) - Ploiesti(9) - Brasov(8) - Targu Mures(16) - Cluj-Napoca(2) - Steiner node - Baia Mare(17).

In order to connect Bucharest to Chisinau, the best way to do it would be to make connections with Iasi(4), even if a straight path would be shorter. So trying to connect all the cities included in this methodology results the next path for this international connection:

- Bucharest(1) - Buzau(18) - Focsani(27) - Bacau(15) - Iasi(4) - Chisinau

Furthermore, this route will be used, as well, for connection with Kiev, the capital of Ukraine.

The third Ukrainian centroid is Odessa, and the connecting links will be passing through some Romanian cities:

- Bucharest(1) - Buzau(18) - Braila(11) - Galati(7) - Odessa.

This should be the best way to connect Bucharest to Odessa and besides that, to introduce links to important nodes from Romania, Braila(11) and Galati(7).

This would be the most important motorway links, which should connect Bucharest with other important external centroids. Of course it is a subjective opinion and different ideas may occur, especially regarding the introduction of few external nodes, namely Lvov and Odessa. Accepting these 8 cities as main abroad traveling directions, the paths of future motorways could be considered the most convenient, especially that are made connections to all the important centroids from Romania, namely the first 10 cities from the list mentioned above.

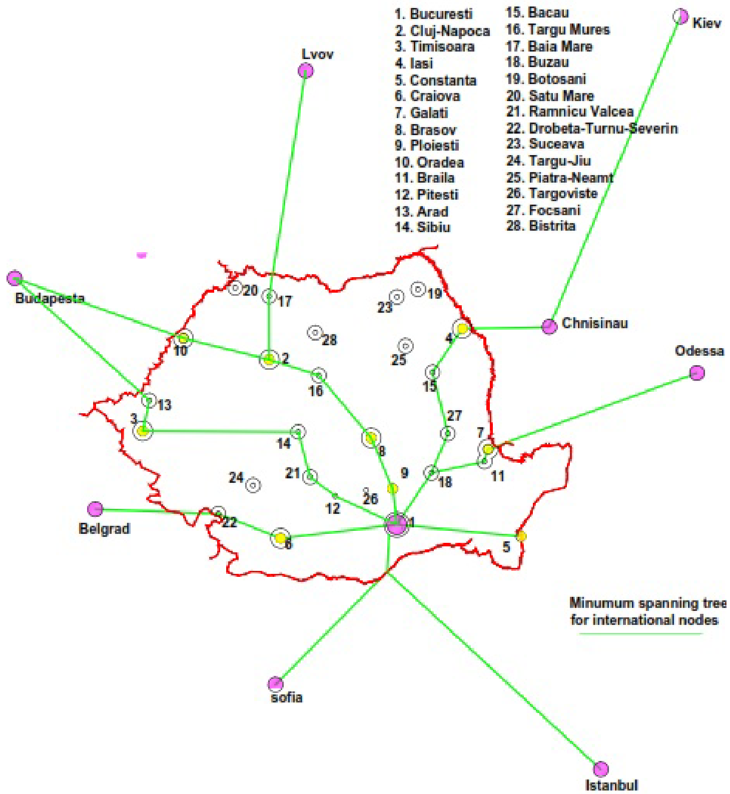

Figure 3 Minimum spanning tree for international connections

After that, new links, between the other cities, are introduced aiming to complete the motorway network.

First of all, as mentioned above a connection between Drobeta-Turnu-Severin(22) and Timisoara(3) will be introduced in order to create a third backup route for the main route Bucharest-Budapest. Also the other cities from Romania, especially from the western part, will have an alternative path to travel to Bucharest.

Also, direct connections should be built from Timisoara(3) to Belgrade and from Craiova(6) to Sofia, making alternative routes to these cities. Also the travelers from the western part of Romania who would like to travel, for instance to Belgrade, will avoid a large detour and will have a more convenient connection.

Another important motorway should be that one who will connect cities from east to west, because the north-eastern part of Romania is not well connected. Even if that most of these links will be very difficult to build because they should cross the Carpathian Mountains, they are very important because there will be a direct link to the other motorways, from the western part of Romania. Also it will be beneficial for tourism. Hence, this motorway will connect the next centroids:

- Iasi(4) - Botosani(19) - Suceava(23) -Steiner Node -Bistrita-Nasaud(28) - Cluj-Napoca(2) - Arad(13).

As it can be seen from figure 4, the centroids from the northern part of Romania are now connected with Bucharest and with the western part cities. Only that they need alternative routes. And for that, a new link has to be introduced, namely on the same direction east/south-west, but connecting another cities:

- Iasi(4) - Piatra Neamt(25) - Targu Mures(16) -Sibiu(14) -Targu-Jiu(24) - Drobet-Turnu-Severin(22).

On this point, it can be said that there are few centroids that are well connected, Bucharest(1), Cluj-Napoca(2), Timisoara(3), Iasi(4), Arad(13), Sibiu(14), Targu Mures(16), Drobet-Turnu-Severin(22). But still, many improvements has to be done in order to create a robust network. For this few links must be introduced on the north-south direction, as follows:

- Constanta(5) - Braila(11) - Galati(7)- Focsani(27) -Bacau(15) - Piatra-Neamt(25) - Suceava(22).

This is the main connection, even if it has some existing links. At this point Romania would almost have a motorway ring road, the missing links being on the western part of the country:

- Arad(13) - Oradea(10) - Satu Mare(20) - Steiner Node - Baia Mare(17) - Steiner Node.

At this point of design we can say that the network looks pretty good. However, two critical points can be seen on the map, namely Brasov(8) and Ploiesti(9), which have only one connection with the network, and are very vulnerable. Also there is still one unconnected centroid, Targoviste(26). For this, two important links have to be introduced:

- Cluj-Napoca(2) - Sibiu(14) - Brasov(8) - Focsani(27).
- Craiova(6) -Pitesti(12) - Targoviste(26) - Ploiesti(9).

Two more links will be added, namely:

- Brasov(8) - Bacau(15).
- Satu Mare(20) - Western border(direction Debrecen)

With this final links added on our ideal network, Brasov(8) becomes a well connected centroid. Actually, this is how the things should be, because is a very important point, a central one, and the core of Romanian tourism.

Of course few links could be added, for instance:

- Iasi(4) - Galati(7), or
- Ploiesti(9) - Buzau(18), or Buzau(18) - Constanta(5),

but these are links that should be built based on concrete surveys, which should justify the future investment. The designing ideal network is based on the size of the cities, and it was conceived on a macroscopic scale, considering only long trips, and assuming that all the travelers will only use the designed motorway network, not the actual national network.

Another matter that should be taken into account is regarding the flow capacity of motorways, but this aspect should be done, as well, based on accurate traffic counts.

An idealistic road network would look like in figure 4:

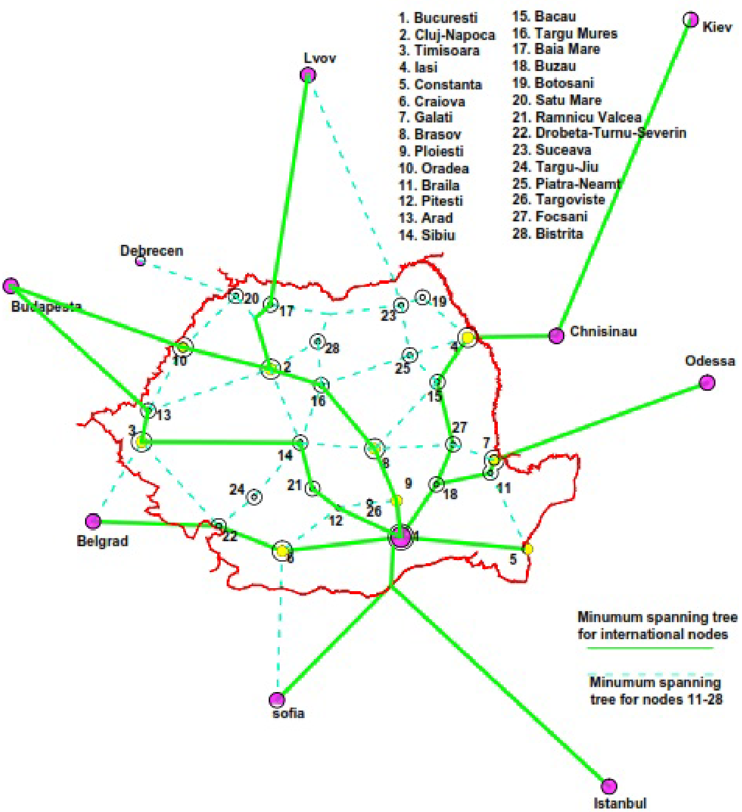

Figure 4 - An idealistic motorway network

5. STEP 4: ANALYSIS OF EXISTING ROAD NETWORK; SITUATION 2012.

At this moment, 2012, Romania has only approximately 500 km of built motorway from a total of 2017 planned km, estimated to be finished till 2020. The main motorways are part of the European corridors IV and IX. These motorways are mostly on the same paths as in the design method. But many improvements have to be made in order to realize a robust road network. Therefore, we will merge the idealistic road network with the real one to obtain the best solution for a robust road network. In figure 5 we can see how the actual Romanian motorway network will look like, and which are the near future plans, for improving it.

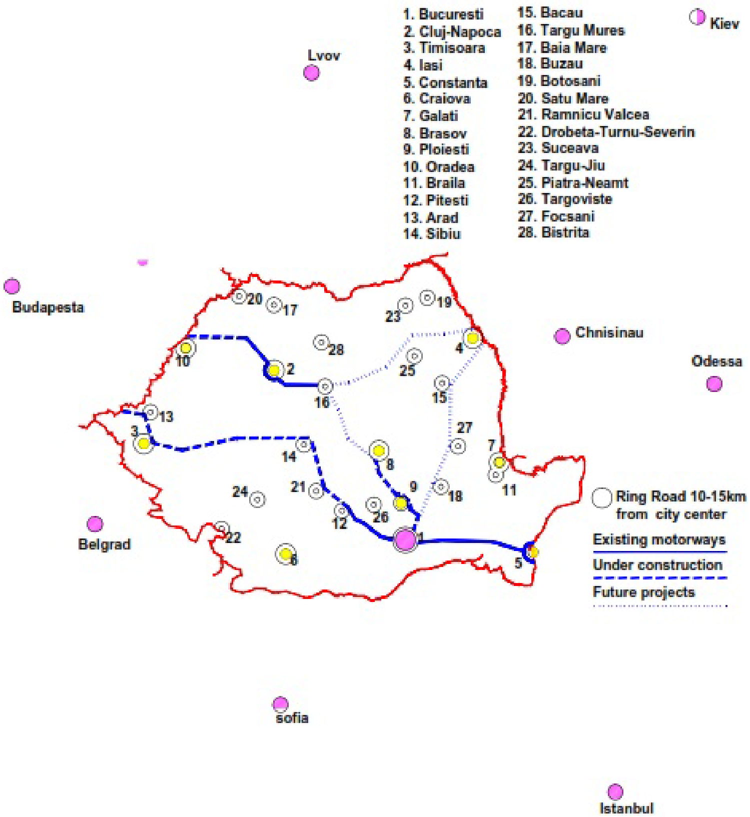

Figure 5 - Existing and future motorways paths

One remark could be made regarding the path of the future motorway from Ploiesti(9) to Buzau(18) ending at a custom village Albita at the eastern border with Moldavia. The proper route for this motorway would be Ploiesti(9)-

Buzau(18)-Bacau(12)-Iasi(4). Thus, two important cities will be connected with the same road sector, and the time travel for the detour for getting on the international path which leads to Chisinau will not be a great one.

Another motorway link will be built in the near future, namely the Bucharest ring road. At this time Bucharest has a ring road, but is not a motorway.

For every centroid, it was considered a ring road at a distance at about 10-15 km from the center of the city. The path of the links were drawn assuming the main traffic flows.

The motorways from abroad were drawn just informational, as a main direction. In reality the Romanian motorways will be connected with the motorways from every country. A combined image of an ideal network and a realistic one is represented in figure 6.

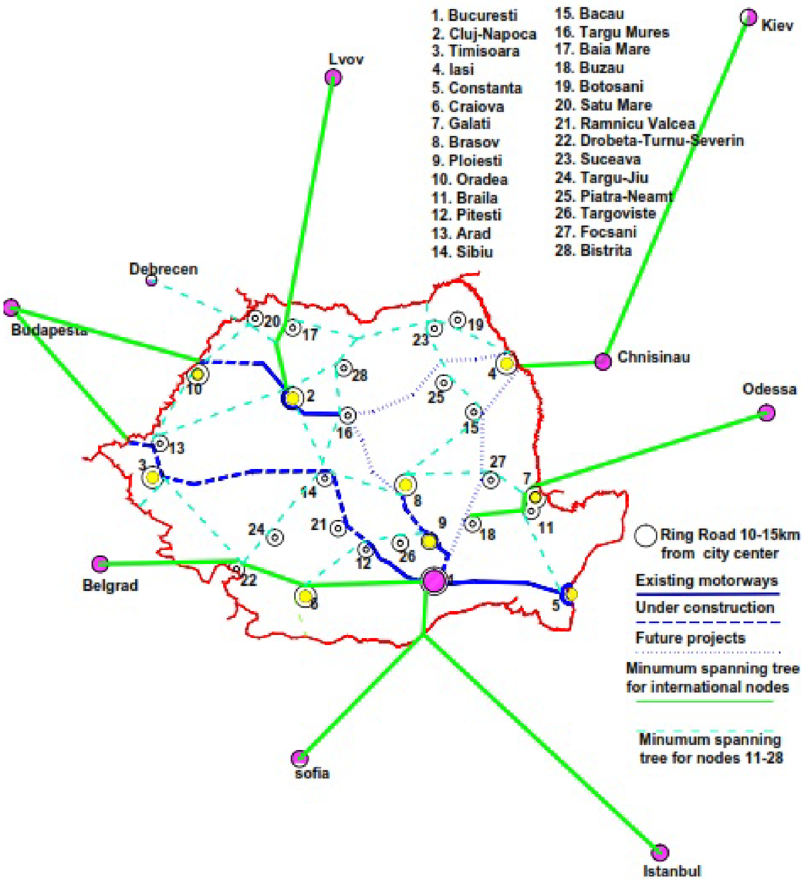

Figure 6 - Final motorway network

3. CONCLUSIONS

This approach of the designing methodology follows only engineering concepts and has nothing to do with political environmental. The fact that the actual motorway network fits almost perfectly with the one from the methodology, is a reason to consider this way of finding the optimal routes for the next paths of motorways, one of the best existing methodology.

Trying to see how efficient this motorway network is, some statistics about the population of Romania who will benefit from the existence of it, are made. Romania has a total population of around 19 million inhabitants, according to the last census. These 28 cities, which were added into the motorway network have a total population of 5.8 million inhabitants, which is 30.53% from the total amount of inhabitants from Romania. Of course this population is only from particularly these cities.

It is sure that we cannot consider this percentage when we refer to the population which is served by the motorways. It will be considered a radius of 30 km from access points, not from the center of cities. We also can assume that, for every motorway link, access points (on and off ramps) will be built at a distances of at least 30 km. So, using this approach resulted a percentage of about 90% (around 17.1 million inhabitants) of population who will have access on the future motorway network within 30 km, or, we may say, in approximately 30 minutes.

Acknowledgements

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References

1. Immers B., Egeter B., van Nes R. - Transport Network Planning: Methodology and Theoretical Notions, Chapter 2, Handbook of Transportation Engineering, Second Edition, 2010;
2. http://en.wikipedia.org/wiki/Pan-European_corridors
3. http://www.recensamantromania.ro/rezultate-2/
4. http://en.wikipedia.org/wiki/Sofia
5. http://en.wikipedia.org/wiki/Istanbul
6. http://en.wikipedia.org/wiki/Belgrade
7. http://en.wikipedia.org/wiki/Budapest
8. http://en.wikipedia.org/wiki/Lviv
9. http://en.wikipedia.org/wiki/Chi%C8%99in%C4%83u
10. http://en.wikipedia.org/wiki/Kiev
11. http://en.wikipedia.org/wiki/Odessa

Risk Assessment for Transportation Infrastructures

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Summary

The paper presents the Risk-Based Estimate Self-Modeling (RBES) which eliminates the need of specialized software by allowing a regular excel user to develop an integrated cost and schedule estimate with limited knowledge of risk analysis. The RBES has the flexibility of entering:

- *The baseline project cost and schedule with different degree of uncertainty associated with them*
- *Simple independent risks with both components: cost and duration*
- *Sophisticated risks which are dependent of each other and/or correlated.*

The RBES integrates the project cost and schedule when the project delivery goes as planned with up to 24 risks events. Each risk may have 2 components: cost, and schedule and the RBES may compute the effect of risks dependency, and risks correlation over the project cost and schedule.

The outcome of the RBES consists of graphs and tables that present the project cost in current year dollars, and year of expenditure dollars plus the Advertisement date, and the End Construction date. In addition to that the RBES computes the tornado diagram of the most significant cost and schedule risks, it presents the risk matrix for each of the 24 risks, and the risks map in order to help the management to understand the project and take adequate response actions.

KEYWORDS: Risk-Based Estimate Self-Modeling, integrated cost, schedule estimate, risk matrix

1. INTRODUCTION

Project cost estimating and risk analysis are important components of project management throughout the life of a project. Good project cost estimating may determine whether the project will go forward or not, and whether its delivery is a success or a failure. Good risk analysis and management may deliver the project on time and on the budget.

There are many methods of developing an estimate for a project. Generally, they are segregated into two major categories:

- Deterministic Estimate
- Stochastic Estimate

The Deterministic Estimate (DE) requires a sagacious estimator who has developed the fine art of contingency’s appreciation for different kinds of situations. The apparent simplicity of the deterministic method is overcome by the following disadvantages:

- It doesn’t promote the identification and management of events (risks) that could change the estimate in a positive or a negative way.
- It allows little control over the project’s estimate.
- It is reactive, and in the majority of cases, any remediation is more costly than it should be.

The Stochastic Estimate (SE) appears more complicated, and it requires quantitative risk analysis that involves team effort and some training in risk elicitation and analysis. The process is quite similar to the “Successive Principle” or “Intelligent Cost Estimating” [1]. The cost of producing SE may be higher than the cost of producing a DE for the same project, but the following benefits of SE go well beyond of a better estimate:

- It recognizes and quantifies the events that may change the project’s estimate.
- It promotes the study of “what if” scenarios.
- It allows reasonable control over the project’s estimate through risk management.
- It shares information among support disciplines.

Furthermore a great benefit of the SE is its richness of data and how the results are organized and inform the decision makers. It gives management users a sharper and far more realistic long-distance view of the prospects awaiting their projects [2].

2. DEFINITIONS

- Base cost -- represents the cost which can reasonably be expected if the project materializes as planned. There is uncertainty or variance associated with the base cost.
- Base variability -- exists even if no risk events are present. Base variability is represented by a symmetric distribution.
- Construction phase – the part of a project life cycle during which the construction work is carried out.
- Correlation –a statistical measure of the internal association between two uncertain variables.
- Current Year dollars – an estimated cost using the time of estimate prices.
- Dependency – establishes relationships among events occurrence.
- Preliminary engineering phase -- the part of a project life cycle during which the planning, scoping, and design phases are carried out.
- Distribution -- a statistical pattern of occurrences of values for a particular outcome when repeated many times. Normal, uniform, pert, and triangular are recommended distributions.
- Escalation – the annual rate of change in cost of the work or its sub-elements.
- Expert judgment -- the judgment provided based upon expertise in an application area as appropriate for the activity being performed.
- Mitigation – an action taken to reduce the impact of an undesirable risk event.
- Monte Carlo Method -- computes the project cost or schedule many times using input values selected at random from the distributions of possible costs or durations and calculate a distribution of possible total project cost or completion dates.
- Probability -- estimates the likelihood that a particular event will occur, usually expressed on a scale of zero to 100 percent.
- Right Of Way – the effort of securing the rights of building a project
- Risk -- uncertain event and its consequences. A positive consequence presents an *opportunity*; a negative consequence poses a *threat*.
- Value engineering – the effort of identifying and removing unnecessary costs.
- Year of Expenditure dollar – an estimated cost of the project when it is anticipated to be built.

3. STOCHASTIC ESTIMATE FUNDAMENTALS

The Stochastic Estimate described by Figure 1 has emerged as a cutting edge estimating method. This concept was created and developed within Washington State Department of Transportation (WSDOT) throughout the progression of the Cost Risk Assessment (CRA) process. WSDOT has more than seven years of institutional SE experience and has conducted hundreds of SE workshops with project values ranging from 1$M to 5$B [4].

The SE is a team approach where a dedicated group meet in a workshop setting and evaluate the project, looking at all phases of project delivery [3,5]. The process is designed to help the project team in its estimating and risks identification process and should not be considered an audit [6].

The SE workshop members review the estimate prepared by the project team; risks are elicited and recorded; project cost and schedule risk analysis is performed; and, finally, results for cost and schedule are presented as graphs, tables, and narrative. Often times risk mitigation strategies are identified during the workshop. The risk analysis presents risk sensitivity results, which allow the project team to develop a sound risk management plan.

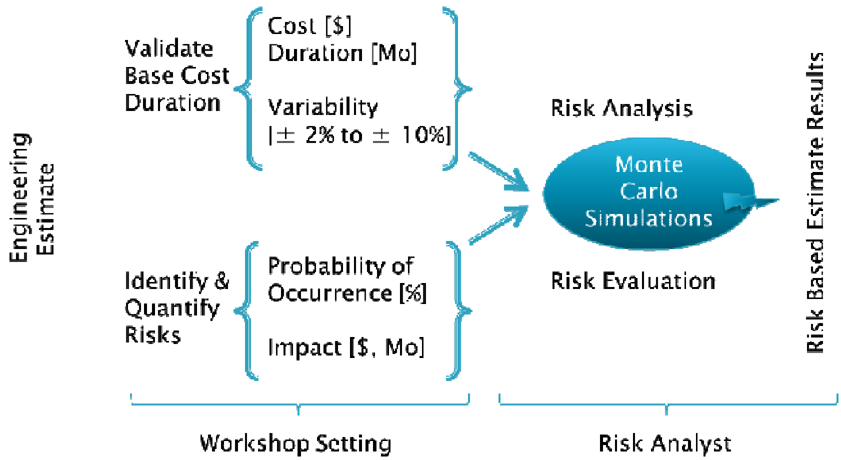

Figure 1. Schematic of the Stochastic Estimate

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4. BASE ESTIMATE REVIEW

The review of the base estimate is conducted by the cost lead, who is an experienced estimator with no stake in the project. The estimator's objective in performing this task is to compute an estimate for neutral conditions. It is critical that the values of the base cost schedule estimate be as accurate as project conditions warrant. The base estimate represents the cornerstone of a project's estimate, and any error would induce linear errors in the project's total estimate.
The review process has the following main steps:

Step 1 Review and validate the project assumptions, and examine, discuss, and document the estimate basis. The outcome of this step will constitute the foundation of risk elicitation.

Step 2 Review the project cost and schedule based on the information available.

Step 3 Removal of contingencies which are hidden or included in various items. The group would focus on removing all contingencies included to cover "what if" scenarios.

Step 4 Capture the "unknown cost" of miscellaneous items, which are usually called "design allowances." This step covers the cost of items that are included in the project but, at the time of estimate, there is no data about how much they might cost [8].

Step 5 Assign variability to the base estimate. The neutral effect of variability should be preserved

5. IDENTIFY AND QUANTIFY RISKS

Risk's identification and quantification is perhaps the most demanding phase of the entire SE process. During this process, the risk lead is responsible for conducting elicitation of unbiased risks and must be aware of numerous biases and agendas some team members may have. The risk elicitor must have the ability to encourage the analysis group to identify all the significant sources of uncertainty and to classify them in sufficiently independent groups [3].
A good risk analysis requires no more then 20 critical risks to be analyzed. At the start of the risk elicitation, limits should be established on what is a significant risk. Typically, there will be only 10 to 20 critical items, even in the largest projects with hundreds or thousands of components to consider [2]. If no critical items are ranged, the inevitable result will be a far narrower predicted range of possible project costs than actually exists; misstatements of risk and opportunity; and understatement of required risk reserve [2].

6. COSTS AND SCHEDULE: INTEGRATED VS. NONINTEGRATED

The simplest SE computes the project's cost using only Current Year (CY) dollars. The project cost could then be aged using different methodologies and could estimate the cost in Year of Expenditure (YOE) dollars. This approach offers relative simplicity, but it lacks the value brought in by integrating cost and schedule.

The integration evaluates in detail the schedule's effect on the YOE estimate and it provides better information to the decision makers. The paper uses an integrated cost and schedule approach where risks affect the CY cost and schedule simultaneously.

6.1. Project's Flowchart

The project's schedule may have different forms and different complexities, and the SE may be applied to any of them. However complex schedule could confuse the workshop participants and for sure will dilute the results.

The simplest flowchart diagram that may be created to represent a project schedule for heavy construction would have four activities. Figure 2 represents a schematic of how the data created for the risk analysis is organized. Each activity has a base cost and base duration assigned.

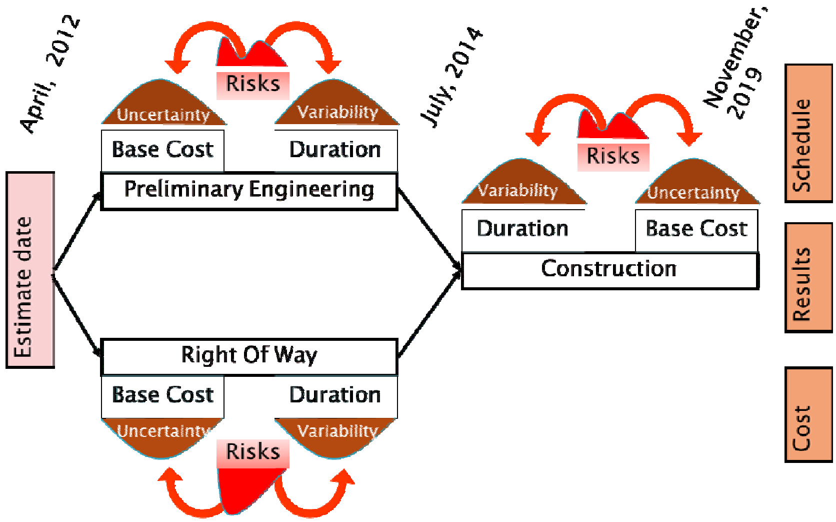

Figure 2. Risk Loaded flowchart

The base cost and base duration are considered in the form of symmetrical distribution and above them risks may affect an activity's cost or duration according to their description.

6.2. Monte Carlo Simulation Applied to SE

Monte Carlo Method (MCM) is a well-established statistical method that consists of running a large number of plausible cases (called iterations or realizations), developing a database, and treating it statistically.

The Figure 2 and Table 1 show how MCM is applied to SE. For a plausible situation the model extracts a random value for the base cost of the Preliminary Engineering (PE), Right of Way (ROW), and Construction (CN) accordingly to their cost distribution.

Table 1. How MCM Works

Phase	PE	ROW	CN	PE	ROW	CN	Total cost
Range	2.5 to 3.5	11 to 13	32 to 35	.5 to .8	1 to 5	2 to 8	
Percentage	100%	100%	100%	50%	30%	67%	
Iteration #	Base cost			Risks Cost when occurs			
1	3	11.6	33	0.7	4	6	58.3
2	2.9	11.9	34	0.55		3	52.35
3	2.8	11.3	33.5				47.6
4	3.4	12	32.9	0.65	3		51.95
5	3.2	12.3	34.6	0.63		7	57.73
6	3.3	12.7	33.5		2		51.5
7	2.8	11.8	33.8	0.66		5	54.06
8	2.6	12.4	32.9			6	53.9
9	3	11.7	34.3			7	56
10	3.3	12.6	34.2	0.76		4	54.86
11	2.9	12.4	33.4		4		52.7
12	2.9	12	33.8	0.65		5	54.35

Total cost	Min value	47.6
Total cost	Max value	58.3
Total cost	Median value	54.0

The model then tests each risk applied to the matching flowchart activity. If a risk was meant to occur, the model will extract a random value from risk's distribution that describes its impact. All of these values are then added together and will form the total cost of the project in CY dollars.

The preceding description is simplistic, but it has the benefit of being intuitive and easy to understand. Reality is more complex when the model must consider all possible situations, such as: risk’s dependency, risk’s correlation, escalation factors, and the schedule’s dynamics, in order to define CY and YOE estimates.

7. RISK-BASED ESTIMATE SEFL-MODELING (RBES) SPREDSHEET

The RBES spreadsheet is designed to facilitate the integration of the cost and schedule estimate of projects by performing Stochastic Estimate [10]. The RBES employs only excel functions, and any user with minimal excel knowledge may use it. It was designed to facilitate SE for projects described by the flowchart diagram, presented in Figure 2.

The RBES is capable of capturing, analyzing, and displaying the simulation results of “pre-mitigated” and “post-mitigated” scenarios on the same graph. The pre-mitigated scenario represents the range and shape of a project estimate before any action is taken to manage risks. The post-mitigated scenario represents the project estimate after the risk management plan is developed. The post-mitigated scenario considers so-called “residual risks,” which remain after the risk response strategy is implemented.

7.1. Base Cost and Schedule Data Input

The base cost and schedule values are entered in the base sheet (see Figure 3). The light-yellow-shaded cells are required to be filled in by the user. The required data should be free of errors because the cells’ definitions are self-explanatory and a pop-up window provides additional guidance if needed.

The RBES is set up to consider the cost in millions of dollars ($M) and duration in calendar months. The Base estimate requires entering the reviewed base estimate values and the base variability requires a single percentage.

The base cost and schedule may be entered for pre-mitigated and/or post-mitigated scenarios. The base sheet contains two tables: the pre-mitigated base cost data and post-mitigated base cost data. The tables are identical as format.

The most critical data that affect the models results are:

- ♦ Estimate Date –defines the current dollar prices and the starting time for defining the escalation length.
- ♦ Target AD data – defines the proposed date for project advertisement
- ♦ A/B/A Duration – defines the time it take since the project was sent to be advertised and the beginning of construction
- ♦ Estimated CN Duration – proposed time for construction including winter time shutdown and other restrictions

- Estimated PE, ROW and CN Cost – the reviewed base estimate for each of these phases
- Variability – one single number which represents plus/minus percentage around the base value. In this way the model preserves the symmetrical distribution for uncertainty in the base.
- Risk Markup – represents markups applied to the base cost. These markups are applied to all risks that affect the construction phase.
- Variability – one single number which represents plus/minus percentage around the base value. In this way the model preserves the symmetrical distribution for uncertainty in the base.
- Risk Markup – represents markups applied to the base cost. These markups are applied to all risks that affect the construction phase.
- WSDOT uses escalation table build in the model. Other users may enter their own escalation rates. Each phase has its own escalation rate.
- Escalation points – defines the time of escalation. By default is 0.5 which it dictates that the estimate is escalated at the midpoint activities.
- Market Conditions (MCs) – capture the possibility of having different bidding environment than what the estimator planned during establishing the base estimate. The MCs may have the most impact on the cost of construction phase and Right of Way phase.

Project Title	Highway to Heaven				Value	Variability	Risk Markups		WSDOT Escalation tables built-in.		
Estimate Date	10/10/08		Target AD date		10/10/09	10%	Mob	10.0%	A/B/A Duration		2Mo
Project PIN #	xxxx		Estimated CN Duration		12.0Mo	15%	Tax	9.0%	Non-WSDOT rates		YOE
Last Review Date	12/12/08		Estimated PE Cost		20.00 $M	5%	CE	8.0%	PE	3%	
Project Manager	JD		Estimated ROW Cost		31.00 $M	20%	PE	9.5%	ROW	5%	
			Estimated CN Cost		210.00 $M	10%	C.O.C	4.0%	CN	2%	
Escalation Points			Base CN Cost Market Conditions			Distribution Type		Base ROW Cost Market Conditions			
Define escalation point of the activity cost. For example 50% means that the escalation point for that activity is the mid-point activity. 50% is the default value. If it is decided that the escalation point is at three quarters of respective activity			Better than planned	10%	30%	PE Duration	U	Better than planned		10%	50%
			Worse than planned	10%	30%	CN Duration	U	Worse than planned		10%	50%
				Probability	Impact	PE Cost	P			Probability	Impact
Preconstruction activities (ROW and PE)		0.5				ROW Cost	T				
Construction		0.5				CN Cost	T				

N=normal distribution; **U**=uniform disgtribution; **P**=PERT or Beta3 distribution; **T**=triangular distribution.

Figure 3. Base cost and schedule data entry

The MCs are described by two components: probability of occurrence and the impact value. Its impact value is entered as a percentage of the activity base cost. For example if the MC is described as 20% probability of occurrence that the bidding environment may drive the cost down by 10%, it means that one out of five time the model will reduce the base cost by 10%.

- Distribution type: defines for each base component the type of distribution. The model can take four types of distributions:
    - Uniform – "U" the base has equal chance to take any value in the range. No clue distribution.
    - Triangular – "T"
    - Pert or beta3 – "P"
    - Normal – "N".

7.2. Risk Data Entry

The RBES allows the entry of 24 risks and each risk may have both components: cost risk and schedule risk. The figure 4 shows how a risk may be entered into the model.

The model allows the analysis of simple cases when risks are considered independent and non-correlated. In this case many of the entry sheet boxes will stay blank.

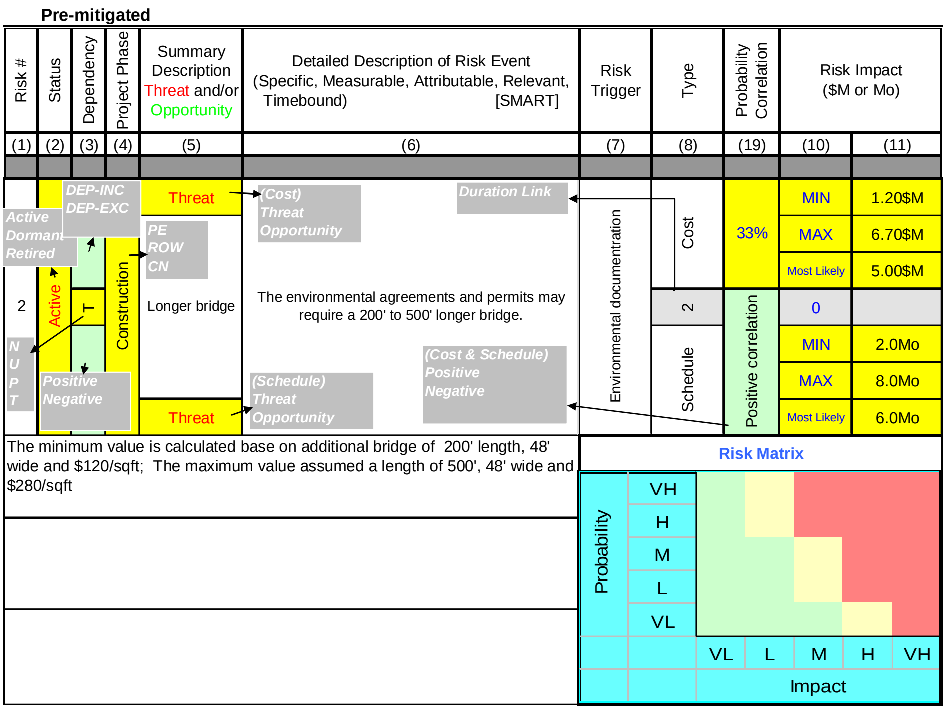

Figure 4. Risk Data Sheet

For more experienced users the model includes advanced futures which allow running of more complex simulation. In this case the user must specify the nature of risks dependency, risks correlation, and dynamics of schedule risks.

7.3. Cost and Schedule Results

The results are presented in the form of graphs (histogram, cumulative distribution function, tornado, and matrixes) and tables.
There are 12 tabs that present the following results:

- Candidates for mitigation in the form of tornado diagrams (1 tab):
- Project's risks map (1 tab)
- Schedule range (2 tabs)
  - Ad Date
  - End Construction Date

- Cost range in CY (4 tabs) and YOE dollars (4 tabs)
  - Preliminary Engineering
  - Right of Way
  - Construction
  - Total Estimate

The estimated cost may look like Figure 5. The distribution is represented by the histogram and cumulative distribution function. When the analysis includes numerous risks the shape tends to move toward a bell curve and the histogram is loosing its identity.
A good risk analysis will have its results individualized. It's possible that the histogram may say its story. If the histogram has 2 or 3 humps it is and excellent display of how a risk or a group of dependent risks impact the project.
The results of the cost distribution or schedule distribution may be used very efficiently for "what if" scenario. The user may re-run the model by retiring a risk or group of risks and examine the results.

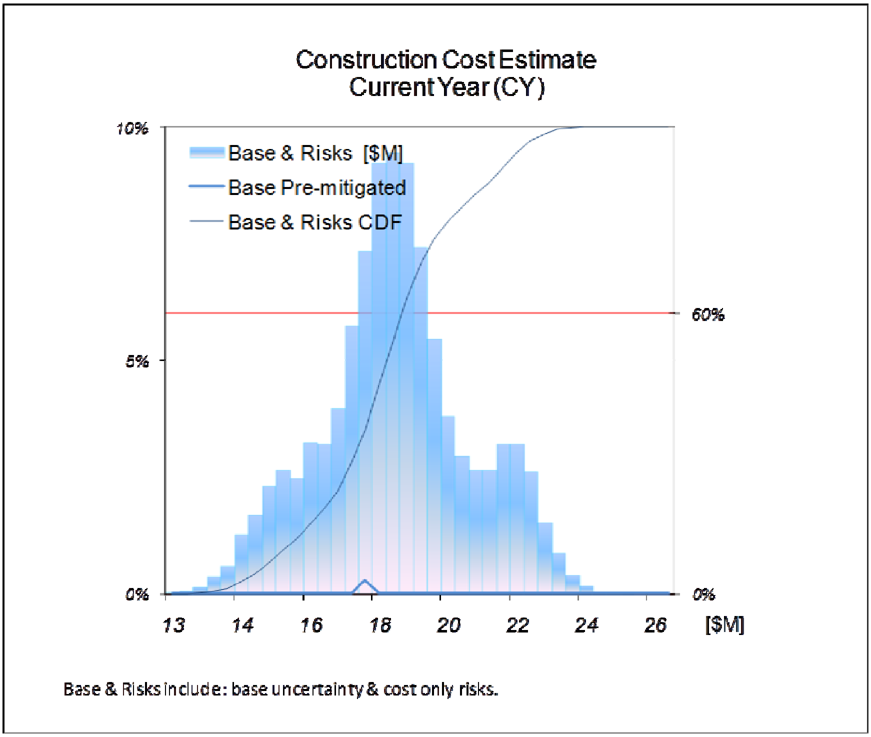

Figure 5. An example of Cost Distribution

Furthermore the model may allow the user to run “what if” scenario by employing the post mitigated future.

The figure 6 shows the situation when the pre-mitigated scenario contains a threat with 50% chance to occur and large impact. The project manager may respond to this risk by spending five millions dollars in base cost and eliminating the risk.

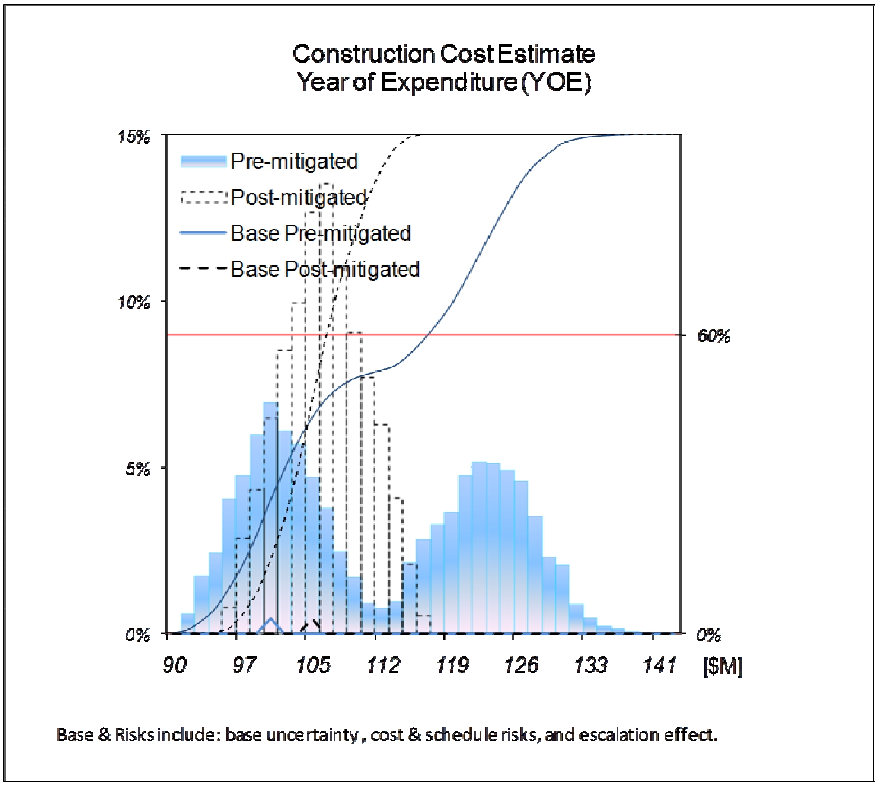

Figure 6. Pre-post-mitigated Histograms

Moreover, large value of variability in the base may hide the risk effect and the histogram may looks too close to a normal distribution. When the base variability has large value the risks' effect may be hidden in the overall variability envelope. Some professionals call this overwhelming effect of variability in the base on results as "induced noise." The risk analysis is about establishing as clear as possible the elements that impact the project and as much as possible their impact. So care should be taken when the variability in the base is established.

A typical "Candidates for Mitigation" (tornado) graph is presented in Figure 8. Each bar of the tornado diagram represents the product of the risk's expected value and its probability of occurrence. The threats are directed toward right and opportunities are directed toward left. It is important to differentiate threats and opportunities because the risk response to each of them is different. Minimize the threats and maximize the opportunity on both characteristics: probability of occurrence and impact.

The tornado diagram gives useful information about the risk's average magnitude but it may mislead the reader on risks with high impact and low or very low probability of occurrence. These types of risks are probably more dangerous than the high probability of occurrence risks since the manager may easily ignore them and when they occur the impact may be dramatic.

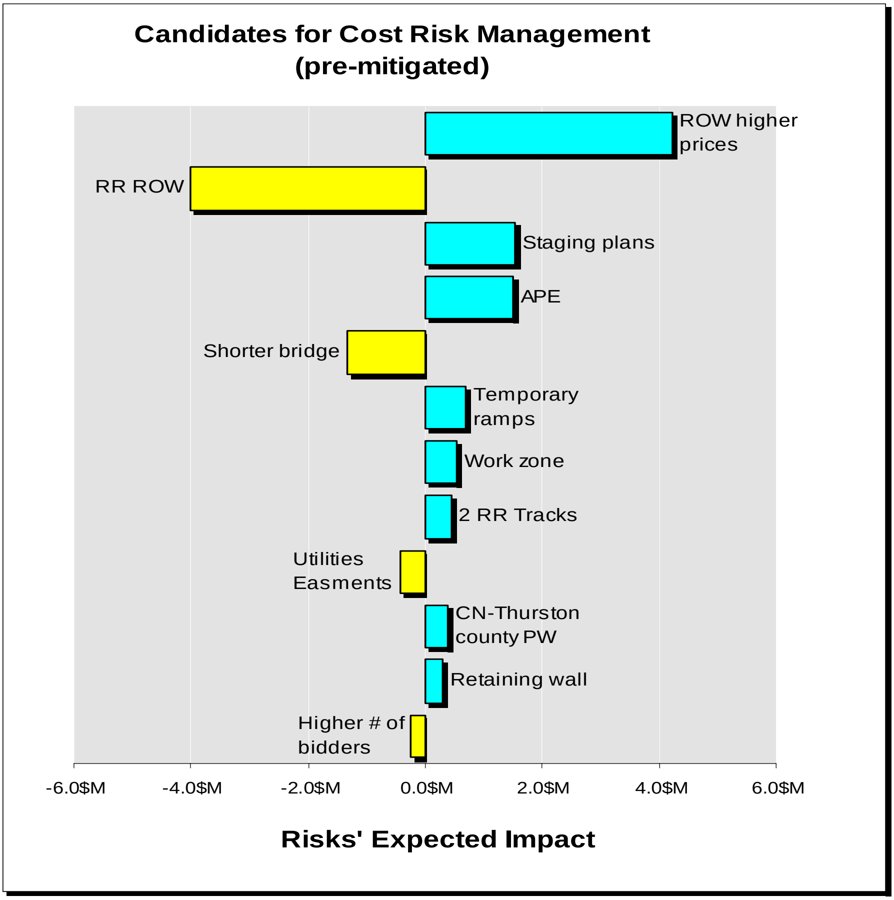

Figure 7. An example of Tornado Diagram

To visualize this situation each risk may be plotted on Risk Matrix (see figure 9). The Risk Matrix is a 5x5 array where the vertical axis represents the probability of occurrence and horizontal axis represents the impact. The cost risk is shown by “$” and the schedule risk is shown by “Mo.”

The shading (color) suggest different level of priority for risk response. The shading closer to the up-right corner represents the first priority on risk response. The shading closer to the down-left corner represents the last priority for the risk response.

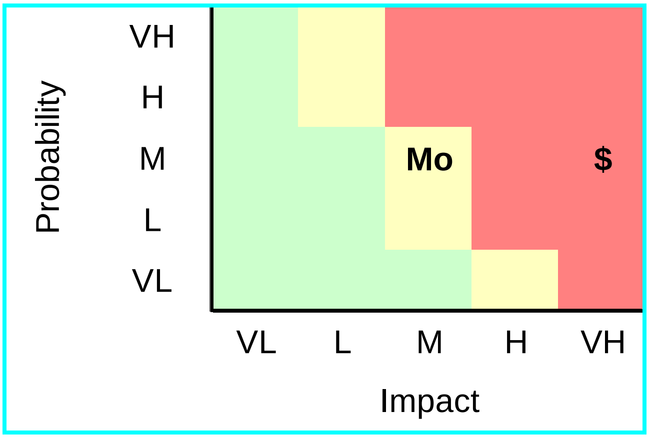

Figure 8. Risk Matrix

A different way of displaying the risk response priority may be provided by the project's Risks Map presented in figure 10.
The projects Risks Map bring together all significant project risks presenting both components: cost, and schedule.

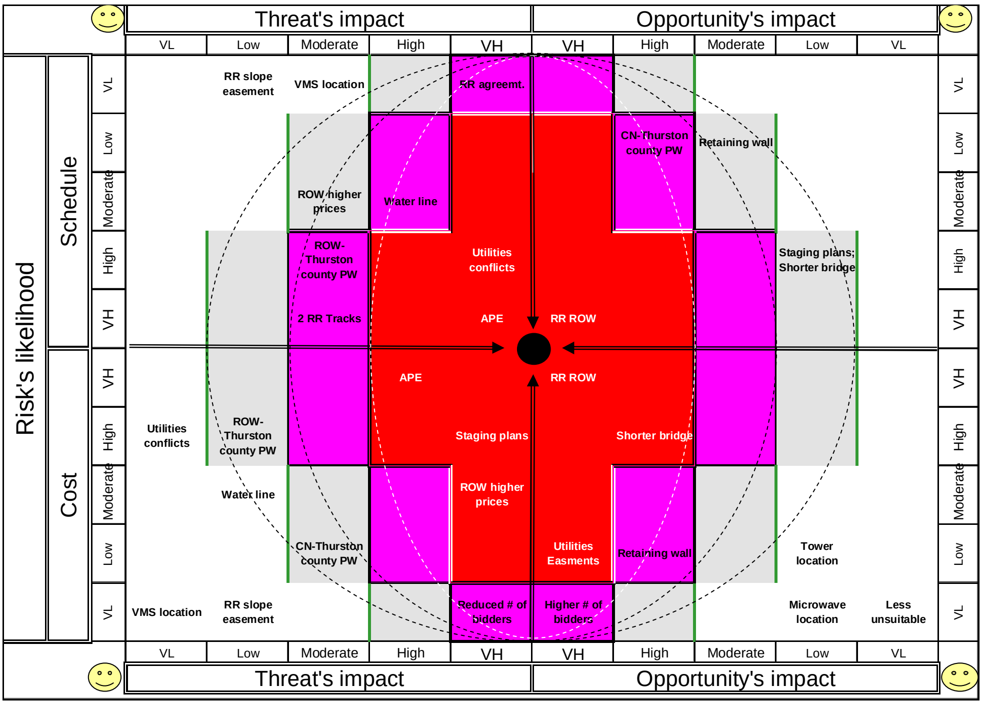

Figure 10. Project Risk Map

The low probability and high impact risks are better represented so the project manager may be alert of what the future may have in store. The project Risks Map

has four quadrants in order to differentiate threats and opportunities and cost and schedule.
As a general rule it is recommended that the risk response strategies should be applied first to risks located in proximity of the map's center (VH, VH). In addition to the color code (shading in black and white) the broken line ellipses complement the order of importance of risks included in each perimeter.
The color code and ellipses emphasize a secondary recommendation: the risk impact is more critical than the risk probability of occurrence. The secondary recommendation makes the broken lines curve to be ellipses and not circle as the general rule will suggest.
Since the risk's impact is more important, in making a decision to respond to that risk or to let it go, than the risk's probability of occurrence the ellipses short axis are horizontal. In this way risks with high impact value and low probability of occurrence will be recognized and perhaps will receive the attention needed.
For example figure 10 shows three events that have very low probability of occurrence with very high impact (RR Agreement, Reduce # of bidders, and higher # of bidders.) The project manager should pay attention to those three events since if any of them happened the project will be significantly affected.
All risk analysis findings are developed to support the project manager in his/her decision on how the project is delivered. The results of risk analysis should be careful evaluated and understood by the decision maker before a decision is taken.

8. CONCLUSIONS

The SE is a valuable estimating process that may assist the project manager with risk management and project cost and schedule estimate. The SE gives management a sharper and far more realistic long-distance view of the prospects awaiting their projects. Through its "Candidates for Mitigation" and "Project Risks Map" the SE provides excellent data for developing a sound project risk management plan.
The RBES spreadsheet is an estimating tool that is useful in implementing the SE in the heavy construction industry, especially in transportation infrastructure cost and schedule estimates.
Also the RBES spreadsheet is a good tool to run "what if" scenarios. It helps project managers keep their estimates up to date and on budget.
The SE and its tool, RBES, may also bring value to Value Engineering studies when it is applied to the "original design" and the "proposed design."
Finally, having the information on pre- and post-mitigation on the same graph gives the viewer a powerful image that could greatly improve the understanding of the project's challenges, and it provides decision makers new, richer data on which to base their decisions.

References

1. Flyvbjerg B., June 2004, "Procedure for Dealing with Optimism Bias in Transportation Planning," British Department for Transport, Guidance Document.
2. Flyvbjerg B., Holm M., and Buhl S., Summer 2002, "Understanding Costs in Public Works Projects," APA Journal, **68**(3), pp. 279-295.
3. Humphreys K., 2008, "Risk Analysis and Contingency Determination Using Range Estimating," AACE International Recommended Practice No. 41R-08.
4. Lichtenberg S., Oct. 2005, "How to Avoid Overruns and Delays Successfully – Nine Basic Rules and an Associated Operable Procedure," ICEC Internet Journal, **18**(4), http://www.icoste.org/Roundup0406/Lichtenberg.pdf
5. Project Management Institute, 2004, "A Guide to the Project Management Body of Knowledge." PMI, 3rd Edition, ISBN 1-930699-45-X.
6. Reilly J., McBride M., Sangrey D., MacDonald D., and Brown J., Spring Summer, 2004, "The Development of a New Cost-Risk Estimating Process for Transportation Infrastructure Projects" Civil Engineering Practice, **19**(1).
7. www.cretugroup.com

The world of bridges: meaning and expectations

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Summary

The article presents two fundamental aspects. On one side the bridge as an object, thing, sign, analized through the prism of semiotics concepts, and, on the other side, the bridge as object of study of stience named bridge engineering, where informatics, systems theory and engineering, reliability, etc. open new frontiers in scientific research of transportation structures.

For bridge analysis a new classification is proposed. This claisfication includes two distinct classes: ordinary bridges and uncommon bridges.

Among uncommon bridges the paper mentions: bridge over Danube River at Giurgeni-Vadul Oii, viaduct "Catusa" at Galati, bridge over Siret River at Galati, etc. This category includes bridges with very large spans, bridges with complex static structures (arches, cable stayed, suspension bridges, etc.) and historical bridges, which survives over centuries (stone bridges: Cârjoaia, Cănțălăreşti, Negoieşti, etc.)

The category of ordinary bridges includes bridges found over all territory of the country. Althou these bridges doesn't impress through technical characteristics they are very usefull to daily human activity.

Bridge engineering is a technical closely related with systems theory and systems engineering. The article proposes that study of the bridges to be accomplished within a sequential concept which includes: design, realization, operational stage, and bridge assessment. This process might be modeled through a closed system represented by the fact that the output depends on inputs, providing that there is at least one input and one output.

The paper presents the pragmatic character of bridge engineering which intends to ameliorate functional and structural design in such a way that the system (bridge) to become optimal from the cost-effectiveness point of view along the operational stage.

KEYWORDS: *bridge, ordinary bridges, uncommon bridges, bridge engineering, ciclurile Ingineriei podurilor, system bridge, object, sign.*

1. INTRODUCTION. THE WORLD OF BRIDGES

When we get into „the world of bridges" we shall aquaint ourselves with famous bridges, miniature bridges and giant bridges, scientists, designers and builders, novelists, poets and artists.

There are covered bridges; others are raising high above waters. There are bridges without which some cities would not be what they are: Venice, London, Sankt Petersburg, Paris etc. There are bridges having extremely long spans; there are famous bridges which many have written about and which many know about.

But in this world of bridges a special place is occupied by the mutitude of ordinary bridges. We see them everywhere around us, located in our familiar cities, integrated into our common roads. The great significance of these bridges can only be appreciated by those people for whom the absence of a bridge can become, in many cases, a real hardship.

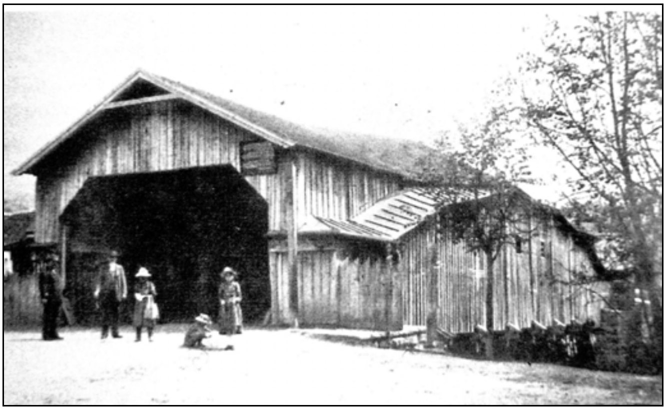

Figure 1. Ordinary covered bridge

There are many who aspired to participate in the achievement of an uncommon bridge, but a few have lived up to seeing thier dream come true. To this end we shall nominate several examples of bridges considered remarkable and which are located in the Romania's geographoic space, without claiming that the list is exhaustive:

- Bridge over Danube River at Giurgeni – Vadu Oii;
- Fetesti – Cernavoda Danube bridge complex on A2 Motorway;
- Bridge over Danube River at Moldova Noua;
- Catusa Viaduct at Galati;
- Grant Overpass, Bucharest;
- Bridge over Siret river at Lutca, a cable stayed bridge;
- Tudor Vladimirescu Bridge and Metalurgiei Bridge, over Bahlui River.

- Bridge over Barlad River at Tecuci;
- Bridge over Lake Mangalia;
- Bridge over Siret River at Galati etc.

The large majority of bridge specialists have dedicated themselves with pertinacity and competence to the multitude of common bridges, by designing them, building them, maintaning them; after a number of years they discovered themselves as being lucky people, a truth confirmed by what Nobel Prize laureate, writer Ivo Andrici wrote about bridges: *„… out of everything that The Man, in his vital propensity, would raise and immure, nothing is more praiseworthy than the bridges; … everywhere in the world, my thoughts are encountering faithful and silent bridges, similar to man's eternal and inexhaustible desire to unite, to reconcile and steadfastly link everything that shows up to human eyes and spirit"*

One should understand the division of bridges into two categories, i.e. common bridges and remarkable bridges, as a flexible classification in agreement with what Blaise Pascal acknowledged: ***„Everything that is perfected by progress disappears by progress"***. Consequently, many of the bridges considered remarkable at a certain time, will be incorporated in the large mass of ordinary bridges. As it is known, exceptions strengthens the rule; herein below several examples of bridges are given, which due to their age, importance and role played along the years can be currently considered remarkable bridges, true monuments which belong to the engineering culture of our nation:
stone bridge in Carjoaia village, Cotnari township;
stone bridge in Negoiesti, Stefan Cel Mare township, known in the literature as Stefan The Great's bridge at Borzesti, the settlement where the Great Sovereign Ruler was born;
Stone bridge in Zlodica village, Cepelnita commune;
stone bridge in Cantalaresti village, built by Goci ataman, brother of Ruler Vasile Lupu.

Figure 2. Stone bridge at Cârjoaia, Hârlau

2. THE BRIDGE

A Bridge could be regarded, interpreted or understood on the basis of the level of education of each of us. It happens that, at a certain moment in time, we find ourselves on a certain step of knowledge, and, given the pragmatism, do not have the capability and desire to climb the steps of evolution. But, many times, our inability to understand some of the phenomena characterizing the construction or performance of bridges should be searched in the way we receive messages through our professional-informational channels, and in our capability of interpreting them.

It is obvious that the concepts of message and information are distinct. It is known also, that the information contained in a message could be more or less interesting, possibly not interesting. The information is disciphered from a message by means of an interpretation rule, called ciphre or key.

In time, for the transmission of messages and information, people have used *long duration bearers*, like stone, clay plates and papyrus. Another bearer of this category is the paper. The messages recorded on paper are the manuscripts, drawings, etc. We shall add to this category of bearers the hard disc, CD, floppy disk or magnetic tape.

Since our *long duration bearers* are the bridges that have been in operation for millennia, the historical information incorporated in an arched bridge stone block, could reach us as a function of our capability to discipher the stone's message, and the motivation of every one of us,

A **bridge** is a construction which has the function of supporting a terrestrial way of communication, ensuring its continuity over a natural or artificial obstacle, fig. 3.

When it is seen, the **bridge** may not be recognized as a bridge. In which case, beyond being subjected to physical interactions, that is beyond the fact that it is a **thing**, it becomes a known **thing**, an element of experience, that is an **object**. But, if it is seen and recognized as a **bridge,** it is not only a **thing** become an **object**, but an **object** become a **sign**. As a **thing**, it only exists: it is a supporting node for a network of physical relationships and actions. As an **object**, it exists for sombeody, as an element of experience; it is differentiating a perceptual thing, by well defined modes, related to its existance as a **thing** for the other elements of the environment.

Figure 3. The Bridge: a thing, an object, a sign

As a **sign**, it is standing not only for itself in the frame of experience and environment, but for something different, something beyond itself. Not only it exists – the thing, not only it is in relationship with somebody – the **object**, but it stays in relationship with somebody, for something different from itself – the **sign**. And this „something different” can or cannot be something real in a physical sense, i.e. what it should accomplish (e.g. to allow crossing of motor vehicles) may not take place, the bridge being damaged.

In this case, its immediate object, the idea which it produces as a sign, becomes in its turn, a supporting node for a network of relationships presumed to be physical, but which, given the fact that the observed bridge could be damaged, are just objectives.

The mode of analysis described above was proposed by Deely starting from an example given by Ch Peirce (who formulated the principles of semiotics). Here we took in consideration the bridge.

Peirce suggested that “it is very easy to see what is the **interpretant** of a **sign**: it is everything what is explicit within the **sign** itself, with the exception of its content and the circumstances of the utterance” [Deely, J]. In the case which we discuss: the **sign** is the bridge, the context and the circumstances of the utterance are the way of communication supported by the bridge, which allows the passing, at a certain moment, of a vehicle on the bridge, and what is explicit within the sign, with the exception of the context and circumstances, is the representation of something different, namely the fact that a vehicle can cross the bridge, although its could be damaged. In other words, everything that is explicit in the sign, except the context and circumstances of the utterance, is its own **significate**, the objective element of the given situation.

The difference between the action of signs and the action of things seems to be clear now: the action of signs is purely objective; it always explains and, simultaneously, exceeds the action of the things as such, while the action of things is purely subjective or, equally similar, is physical or psychic and confined to what exists.

We think that a discussion about the bridge could be extended to another context, namely, by corroborating the bridge, as a thing (concrete, abstract, real), with the physical model (statical schemes, dynamic systems etc) and with the mathematical model (condition of static equilibrium, compatibility of deformation, relationships between forces and displacements etc).

The sign not only stays for something else than itself, but it does that in relationship to a third party, hence the two relationships – between the sign and **significate** and between sign and **interpretant** – can be taken separately. When they are taken this way the question of sign is not raised anymore. Instead the relationship between cause and effect, on the one hand, and between object and expert subject, on the other hand, are raised.

Consequently, for the relationship between the **sign** and **significate** to exist as a semantic relationship (statical schemes, as a sign for the bridge, mathematical model as a sign for the bridge etc), no matter whether this dyadic relationship exists, as a cause-effect relationship (state of deformation of a bridge during the passage of a vehicle), reference made to the future by a tertiary element, the interpretant, is essential. The sign is representative only in a direct way, that is in subordinate capacity. When it becomes autonomous is is just an object, or a thing which became and object. Therefore, a sign is representative only in case it represents something different from itself. „To be a sign is a form of slavery in relation with something else" [2].

3. BRIDGE ENGINEERING

To start with, let us define the *Bridge engineering*. To this end, we resort to our own concepts and methods: Systems theory and Systems engineering. Thus, **Bridge engineering** is a sequential concept which comprises: design, construction, operational stage and structural evaluation. This process may be modeled by a closed system, made concrete by the fact that outputs depend on inputs, in the presence of at least one output datum and one input datum. Fig 6.

Subsequently, *Bridge Engineering* has a strong pragmatic character, inasmuch as it tries to enhance the functional and structural design, so that the system, i.e. the bridge, becomes optimal in respect to cost – efficiency, along its operational life.

In *bridge engineering* the solutions adopted should be sustained by technical and economic efficiency. Efficiency depends on the following factors:

- Materials of which the bridge is built. This intervenes by an efficient conditioning of quality and cost;

- Static, dynamic and seismic analysis. These intervene through the scientific, theoretical and experimental level which forms the basis of official norms that governs the bridge engineering;
- The concept of general and detailed constructive constitution. This factor is mainly determined by the level of professional, theoretical and practical training of the designers;
- The construction technology depends on the level of professional training of the workforce and the equipment of contractors.

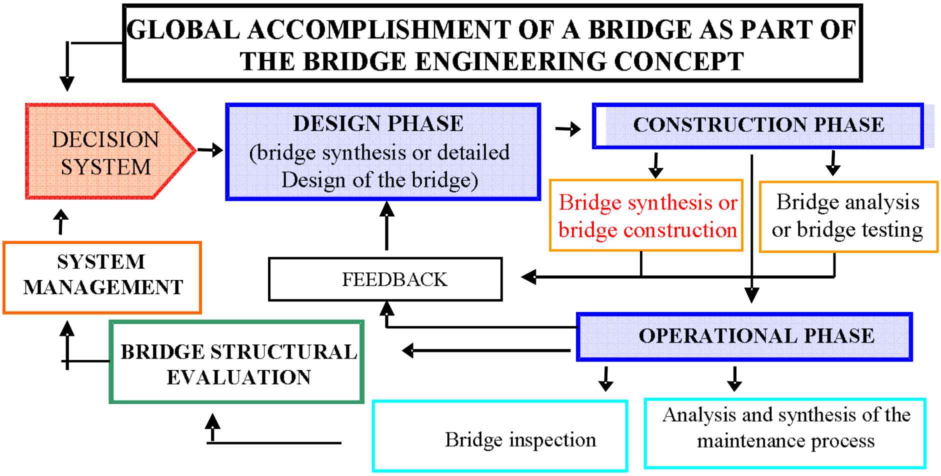

Figure 4. Cycles of bridge engineering

The optimization of the above mentioned factors during the bridge engineering process needs a systemic approach on different levels. For example, a first level could be imagined by harmonizing the information held by the research engineer, design engineer, construction engineer, the expert and the employer. A second level of competence is the one by which the total accomplishment of a bridge by means of bridge engineering cycles is defined, figure 4.

If we find ourselves in the area of the first cycle, namely the design, we distinguish several important factors which influence the experts' expectations to bring about modern and efficient bridges. We refer to ways by which we can develop the conception domain (design = systhesis), by attracting a larger number of authentic specialists and by applying new efficient solutions. It is necessary to lay special emphasis on the use of computer aided design, apply methods based on computer simulations, optimizations, develop construction technologies which would allow mechanization of works etc.

Another important factor is the enhancement of decision mechanisms. The final solution should always be found by a scientific negotiation of all technical aspects which can be highlighted during the global accomplishment process of a bridge.

The information coming from the process of gathering and processing the data specific to bridge construction, operation and structural evaluation phases, are utilized as knowledge in the design phase to envision the future bridges.

Data resulted from activities specific to bridge engineering cycles are evidenced through a gathering process, in the form of factual entities: numerical values, perceptions and observations, that constitute a rough informational material; that could be processed through a transformation method, depending on variuous objectives, figure 5.

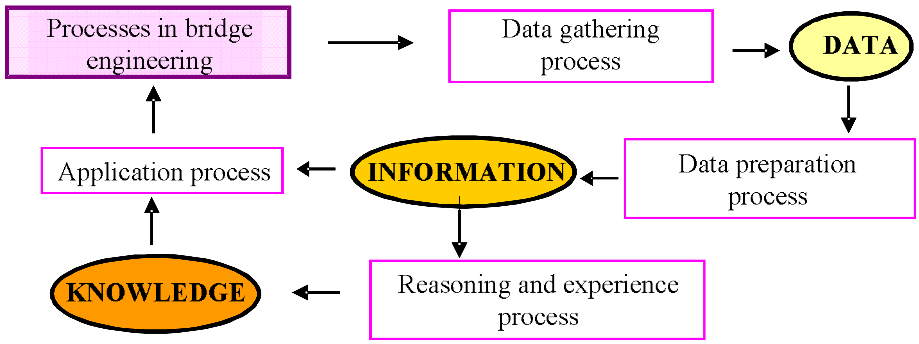

Figure 5. System of data gathering and their transformation into information and knowledge, in the Bridge engineering process

The operational phase comprises two important processes: *operation and maintenance*. *Operation* of a bridge is identified as an operational process by which the bridge is utilized in agreement with the purpose for which it was completed. Depending on the inputs into the bridge-system we distinguish a *continuous sub-process*, generated by the permanent loadings and a *discreet sub-process*, caused by the temporary or exceptional loadings.

Finally, the evaluation of the bridge-system is aiming at utilizing the essential performances of the bridge by means of its technical characteristics and its performance, maintainability and serviceability indicators, the general cost of the bridge-system and its aesthetic performances.

By making comparable the operation of bridge-system evaluation and a sub-system, as a component part of the bridge engineering process, then, this could be systemically tackled by using the concept of *system analysis*, which, as it is known, has the following ways of action: *observation, experiment and reasoning*.

4. MEANINGS IN THE WORLD OF BRIDGES

It is known from mechanics that a segment AB can be traveled in two directions, from A to B and, viceversa, from B to A. A segment which has a direction set on it is called a vector. If the vector is AB, with its sense from A to B, A is called the origin of the vector, and B is called its extremity. The length of segment AB is called the magnitude of vector AB. If A = B, the magnitude of vector, when its unit of measure was set, is a real positive number. Let us agree that, in case points A and B coincide, they form a null vector.

In line with the above it is needed that, in the future, we should avoid confusion between the two concepts: direction and sense, in the mathematical understanding of these notions, remove the cause, internal or external, that would favor elimination of progress, have the magnitude of vector be equal to zero, and have an incorrect selection of the sense. Within everything that happens in such a complex field as structures, the correct sense is established if perfection becomes the guiding idea.

On the other hand, it is known that the concept of vector is formed by a material psychic process, relating to reality phenomena in which the so called vectorial magnitudes occur: we travel, in what direction? How far? Or: a weight is tied to a rope, and we pull, in what sense? How hard?

The obvious question is raised: in order to progress which is the sense that we should follow? We shall find the correct answer to this question only if we place in the center of our practical preoccupations, the fundamentals of Information Theory, Systems Theory, Structures Theory and Reliability Theory.

In the modern understanding of the science, the world of bridges constitutes a system. A system is constituted every time a relationship is highlighted between minimum two magnitudes or objects. Also, the physical achievable model of the relationship y (outputs from the system) and u (inputs into the system) is called a system, if structure S is partially, but sufficiently, known to demonstrate the causality $u \rightarrow y$.

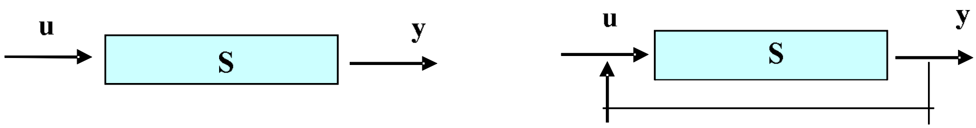

Figure 6. Open and closed systems

The experience accumulated in the system analysis field shows the existence of two fundamental dependence modes between output and input:

- the output is determined by the input, directly, through the state;
- the output depends on the input, but it modifies it to a certain extent.

We call an open system that system in which the output depends on the input, without any reverse connection. A closed system is characterized by the presence of a reverse connection (from the output towards input) between at least one output and one input. The reverse connection is called feedback. In closed systems we are dealing with a control of output upon input.

Most of the times, in a closed system we can identify three sub-systems: *a guided sub-system* (of which output controls the input of the whole system), a *guiding* or *decisional sub-system* (of which output is the input of the guided sub-system) and a *feedback sub-system* (which sends the output of the guided sub-system to the input of the decisional sub-system).

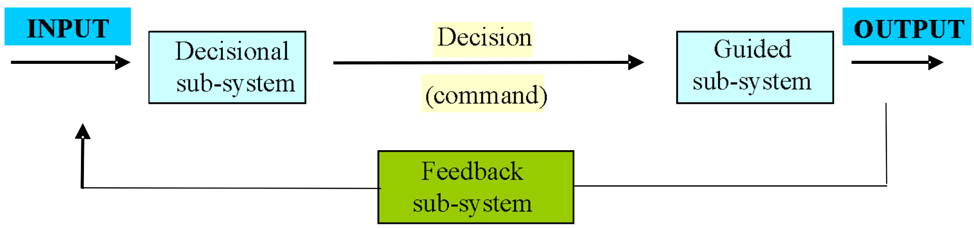

Figure 7. The sub-systems of a closed system

The identification, by the Man, of the reality as a group of objects which interact, the perception of the environment, close up or remote, as a multitude of systems, but especially the knowledge of the structure and performance mechanisms of these systems, have traveled the thorny road of all scientific conquests.

The systems theory had developed on the basis of an accumaulation of facts, information, and data. The information is the essential support of system steering. Any undertaking is constituted in a system and, consequently, at the level of strategic and tactical management, the information is needed both for the formulation of methodologies, for the fulfillment of objectives and the verification of its translation into real life. At the level of operational management information is needed for a detailed planning and for the evaluation of performances.

Consequently, the activity of resolving the complex problems of bridges, forms, in the Systems Theory and Information Theory concept, an information system. If in the framework of an information system the automation means, based on electronic computers, predominate as equipment, this shall be called an information system.

By an information system one should understand the whole complex of activities which record the information, and process it, and use it for accomplishing the objectives of the organization.

An advanced processing of information in an information system is obtained only if we use *databases* and *expert systems*. And, not lastly, a modern orientation in the bridge theory should establish the procedure by which an authentic intellectual, emotional and volitional spiritual mood is created, taking into consideration the traditions, habits, expectations. It seems that the answer to this problem can only be given by The Theory and Computation of Structures, which, as it is known, are forms of scientific knowledge consisting of an ensemble of information and mathematical operations, ordered systematically. It offers both a description and an explanation of structures' performance in time, and the parametric values which intervene in the relationship actions – structure – response. In the modern age which we are now covering, the complex problems presented above will best abide by our will only if we have recourse to appropriate hardware and software configurations, handled competently by properly trained specialists.

5. CONCLUSIONS. EXPECTATIONS IN THE WORLD OF ORDINARY BRIDGES

Let us *enlighten,* with carefulness, *engineering structures* that are related to our society characteristics, by using the experience gained until now.

In the future the concept design should represent the result of confronatations between the *social demand* and the *available resources,* specific, on the one hand, to the phase of development, and to capability of finding solutions to meet certain *performance levels*, at a given degree of meeting the social demands, on the other hand.

We are positive that, at a certain moment in time, a consensus will be reached, on the basis of which the common bridges will be conceived, accomplished, studied maintained and strengthened by means of *specific methods and procedures*.

Theoretical and experimental methods will be crystalized, specific to this type of bridges, which will lead to a significant accumulation of information, and, by a rational and automated processing of them, a transfer of knowledge will be carried out, and, ultimately, to an *improvement by progress*.

Intelectual states of mind will be created at a certain time, which will allow us to search, in the past, many of the ordinary bridges of the future, concurrently with the rehabilitation of two construction materials: *stone* and *timber*, which played an important role for a long period of time in the history.

The approach, in conformance with *Bridge Engineering*, by including principles and concepts from the Systems Theory and Systems Engineering, would widen the theoretical framework of research of a bridge (which will not limit itself only to classical sciences: Statics, Sytrength of materials, Dynamics of constructions, Geothechnics and Foundations), by opening of new horizons towards modern sciences such as: Systems Reliability, Information Sciencs, Technical Systems Modeling and Simulation etc.

Complete computerization of all fields of activities of the bridge specialists will become a requirement: *databases, expert systems and computer programs* will allow us to conduct these activities more quickly and better, and results are better and longer lasting.

References

1. Burlacu, Eugen ş.a., Introducere în studiul fiabilității sistemelor, Editura Ştiințifică, Bucureşti, 1995;
2. Deely, J., Bazele semioticii, Editura All, Bucureşti, 1997
3. Ionescu, I., “The Bridges”, Polytechnic Society Bulletin, Vol I, Bucharest, 1931;
4. Moisil, Gr.C., Lecții despre logica raționamentului nuanțat, Editura Ştiințifică şi Enciclopedică, Bucureşti, 1975;
5. Rafiroiu, M., Modele de simulare în construcții, Editura Facla, Timişoara, 1982;
6. Salvadori, Mario, Mesajul structurilor, Editura Tehnică, Bucureşti,1991;
7. Zadeh, L.A., Polak, E. ş.a., Teoria sistemelor, Editura Tehnică, Bucureşti,1969;

Study for evaluation of pavement technical condition on the county road DJ 247A Iasi - Bârnova for prioritization of the repairing works

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Summary

This paper presents the results obtained from investigation of technical condition of the asphalt pavement on the county road DJ247A sector: km0+000 - km10+000. Intervention strategies obtained by using the Rehabilitation Programme Analysis System - PRAS were described and also a prioritization equation has been used in parallel with the PRAS software.

Finally, a comparative analysis of the prioritization results, obtained by using these two methodologies, is performed.

KEYWORDS: technical condition, asphalt pavement, intervention strategies, PRAS, prioritization equation.

1. INTRODUCTION

For evaluating the technical condition of this road a representative sector of 10 Km length, was picked up on DJ 247A Iaşi -Bârnova.

2. PRAS METHODOLOGY

The Program Rehabilitation Analysis System is specific software, providing specific intervention strategies that must be applied to improve the technical condition of the road.

3. TECHNICAL DATA REGARDING THE ROAD SECTOR

For assessing its technical condition, the road has been divided into three homogeneous sectors , regarding the specific distresses found during the site investigation.

The first sector is located between km 0 + 000 and km 3 + 200 and is characterizesed by the following technical data:
ADT = 10.000 vehicles
The road structure is a flexible one, being made of:
- Foundation layer: 10 cm ballast
- Base course granular : 12 cm broken stone
- Wearjng course : 8 cm asphalt mixture.

The second sector is located between km 3 + 200 and km 5 + 700 and is characterised as follows :
ADT = 5.000 vehicles
The road structure is a flexible one , being made of:
- Foundation layer: 10 cm ballast
- Base layer: 12 cm broken stone
- Wear layer: 8 cm asphalt mixture.

The third sector is located between km 5 + 700 and km 10 + 000 and is characterised as follows:
ADT = 8.000 vehicles
The road structure is a flexible one being made of:
- Foundation layer: 10 cm ballast
- Base layer: 12 cm broken stone
- Wear layer: 8 cm asphalt mixture.

4. INVESTIGATION OF TECHNICAL CONDITION

The results of the field inspection are presented in relation with Table1, Table 2 and Table 3, as follows:

Road sector DJ 247A - Sector 1
Km: 0 + 000 Km 3 + 200
Sector length: 3.200 km
Date of investigation: 25.11.2012

Table 1. The results of site investigation of the pavement condition on Sector 1

Code	Type of degradation /Type of distress	Type of degradation observed	Degrees of severity	The degree of extension	Obs. dv
#1	Aligator cracking	5	0.7 (M)	0.7 (F)	2.45
#2	Bleeding				
#3	Transverse and block cracking	10	0.7 (M)	0.7 (F)	4.9
#4	Bumps and sags				
#5	Corrugations				
#6	Depressions				
#7	Edge cracking	3	1 (H)	1 (E)	3
#8	Joint reflection cracking	3	1 (M)	0.8 (F)	2.4
#9	Shoulder drop off	2	0.7 (M)	0.7 (F)	0.98
#10	Longitudinal and transverse cracking, longitudinal joint cracking, wheel track cracking	20	0.7 (M)	0.7 (F)	9.8
#11	Patching-utility cuts				
#12	Polished aggregates	2	0.8 (L)	0.6 (O)	0.96
#13	Potholes	7	0.7 (M)	0.8 (F)	3.92
#14	Railroad crossing				
#15	Rutting				
#16	Shoving				
#17	Slippage cracking				
#18	Swell				
#19	Wethering and raveling	5	0.6 (M)	0.8 (F)	2.4
				dv =	30.8 1

(*)Note: L–Low; M–Medium; H–High; O–Occasional; F–Frequent; E–Extended;

PCI = 100 – dv = 100 – 39.81 = 69.19
PCI = 69.19

Road sector DJ 247A - Sector 2
Km: 3 + 200 Km 5 + 700
Sector length: 2.500 km
Date of investigation: 25.11.2012

Table 2. The results of site investigation of the pavement condition on Sector 2

Code	Type of degradation /Type of distress	Type of degradation observed	Degrees of severity	The degree of extension	Obs. dv
#1	Aligator cracking	5	0.7 (M)	0.7 (F)	2.45
#2	Bleeding				
#3	Transverse and block cracking	10	0.4 (L)	0.5 (O)	2
#4	Bumps and sags				
#5	Corrugations				
#6	Depressions	10	0.5 (L)	0.5 (O)	2.5
#7	Edge cracking	3	0.7 (M)	0.7 (F)	1.47
#8	Joint reflection cracking				
#9	Shoulder drop off	2	0.4 (L)	0.7 (F)	0.56
#10	Longitudinal and transverse cracking, longitudinal joint cracking, wheel track cracking	20	0.4 (L)	0.7 (F)	5.6
#11	Patching-utility cuts				
#12	Polished aggregates	2	0.8 (L)	0.9 (F)	1.44
#13	Potholes				
#14	Railroad crossing				
#15	Rutting				
#16	Shoving				
#17	Slippage cracking				
#18	Burdusiri/Swell				
#19	Wethering and raveling	5	0.6 (M)	0.5 (O)	1.5
				dv =	17.52

(*) Note: L–Low; M–Medium; H–High; O–Occasional; F–Frequent; E–Extended;

PCI = 100 – dv = 100 – 17.52 = 82.48
PCI = 82.48

Road sector DJ 247A - Sector 3
Km: 5 + 700 Km 10 + 000
Sector length: 4.700 km
Date of investigation: 25.11.2012

Table 3. The results of site investigation of the pavement condition on Sector 3

Code	Type of degradation /Type of distress	Type of degradation observed	Degrees of severity	The degree of extension	Obs. dv
#1	Aligator cracking	5	0.7 (M)	1 (E)	3.5
#2	Bleeding	3	0.8 (L)	0.6 (O)	1.44
#3	Transverse and block cracking	10	0.7 (M)	1 (E)	7
#4	Bumps and sags				
#5	Corrugations				
#6	Depressions	10	0.5 (L)	0.5 (O)	2.5
#7	Edge cracking	3	1 (H)	1 (E)	3
#8	Joint reflection cracking				
#9	Shoulder drop off	2	0.7 (L)	0.7 (F)	0.98
#10	Longitudinal and transverse cracking, longitudinal joint cracking, wheel track cracking	20	0.7 (L)	0.7 (F)	9.8
#11	Patching-utility cuts				
#12	Polished aggregates	2	0.8 (M)	0.9 (F)	1.44
#13	Potholes	7	0.7 (M)	0.8 (F)	3.92
#14	Railroad crossing				
#15	Rutting				
#16	Shoving	2.5	0.6 (M)	0.5 (O)	0.75
#17	Slippage cracking	2.5	0.3 (L)	0.5 (O)	0.38
#18	Swell				
#19	Wethering and raveling	5	0.6 (M)	0.5 (O)	1.5
				dv =	36.21

(*) Note: L–Low; M–Medium; H–High; O–Occasional; F–Frequent; E–Extended;

PCI = 100 – dv = 100 – 36.21 = 63.79

PCI = 63.79

5. IMPLEMENTATION OF THE PROGRAM PRAS

In the following, in relation with various figures, the main stages in the application of the PRAS software, on these three hommogenuous sectors, selected on the DJ 247A, are presented:

DJ 247A - Sector 1

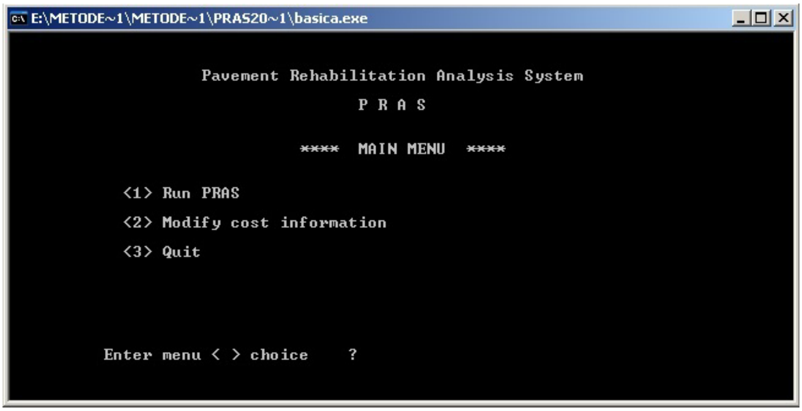

Figure 1. Print Screen with main menu

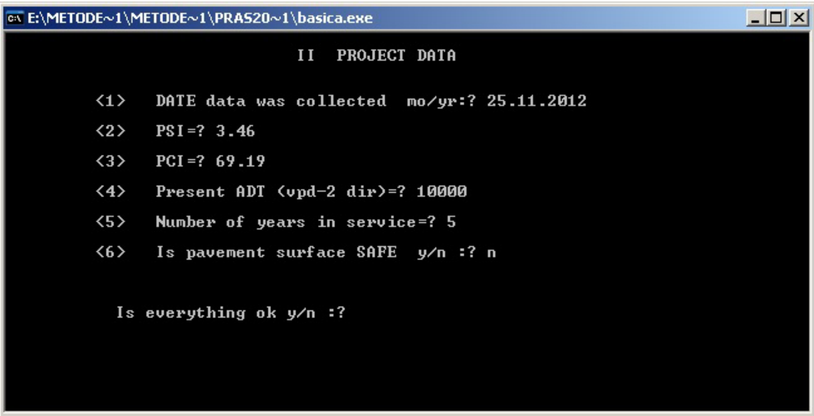

Figure 2. Print Screen with Project Data

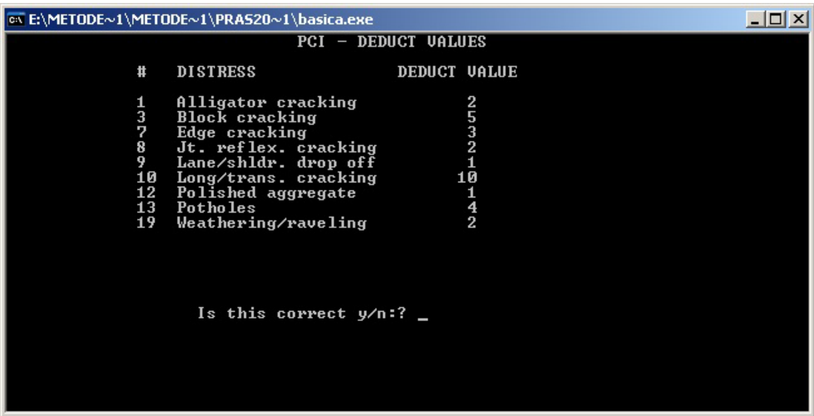

Figure 3. Print Screen with PCI – Deduct Values

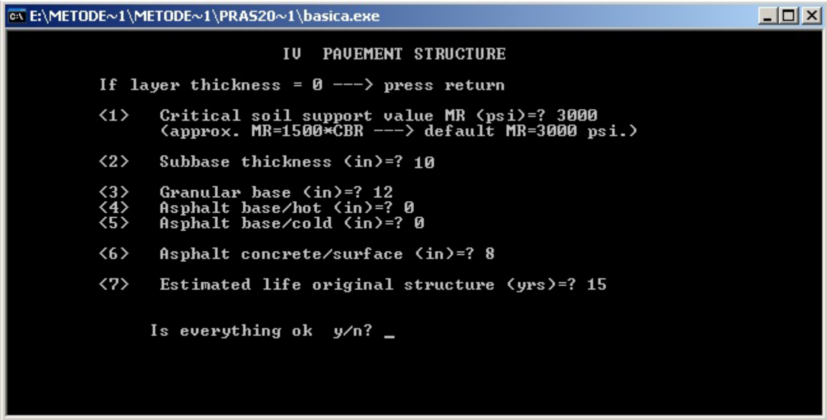

Figure 4. Print Screen with Pavement Structure

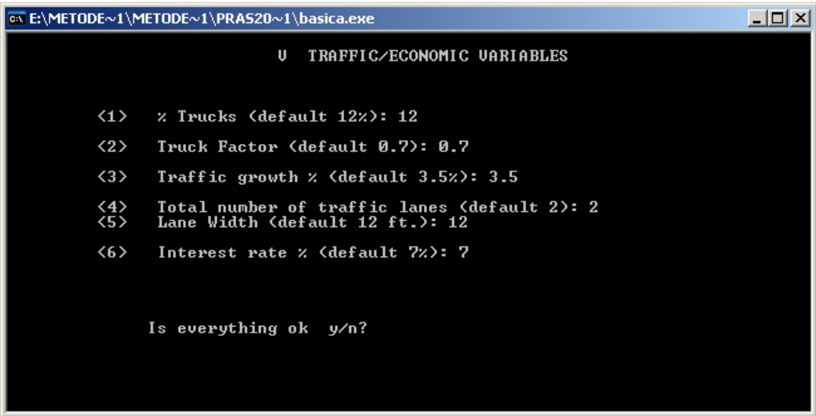

Figure 5. Print Screen with Traffic/Economic Variables

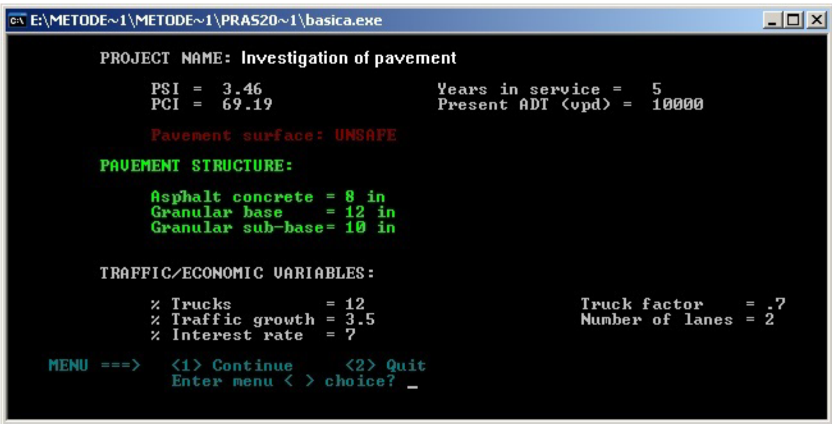

Figure 6. Print Screen with Synthesis of the input variables

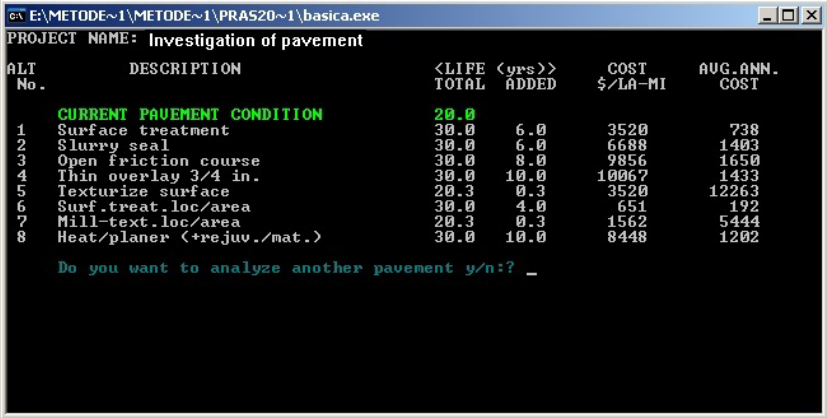

Figure 7. Printscreen with Solutions for road rehabilitation

DJ 247A – Sector 2:

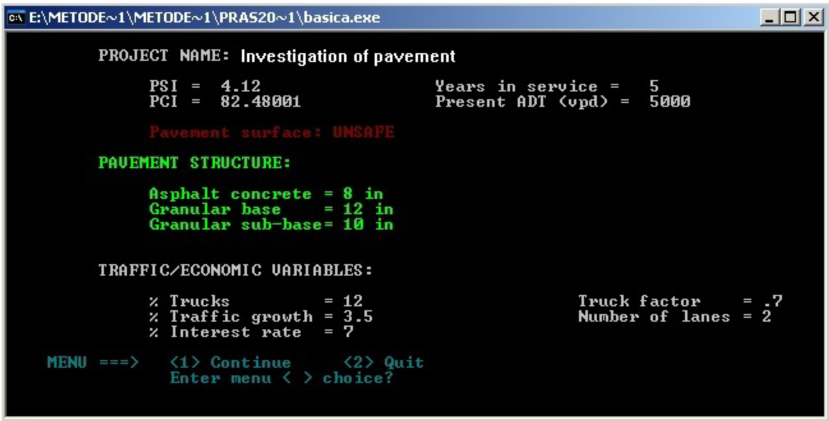

Figure 8. Print Screen with Synthesis of the input variables

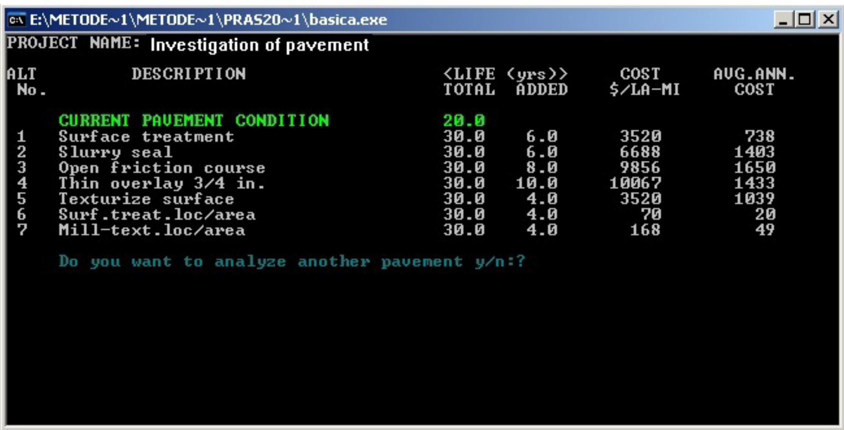

Figure 9. Print Screen with Solutions for road rehabilitation

DJ 247A – Sector 3:

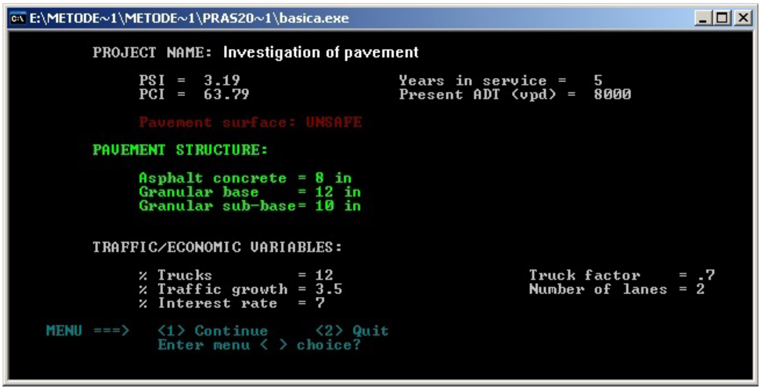

Figure 10. Print screen with Synthesis of the input variables

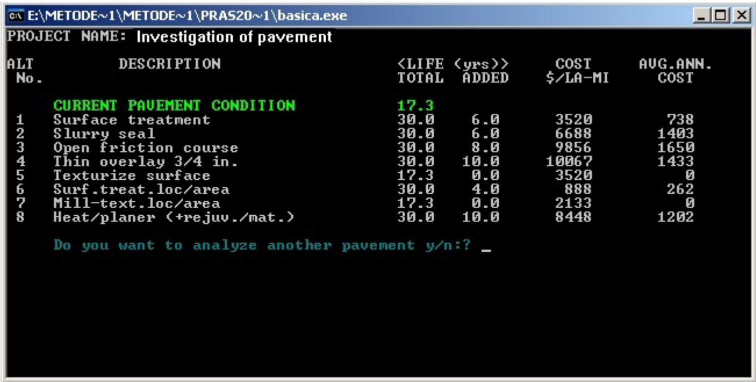

Figure 11. Print Screen with Solutions for road rehabilitation

Table 4, presents a synthesis of the application of PRAS software on three sectors.

Table 4. Results obtained using PRAS program

Sector	PSI	PCI	ADT	No. strategies
S1	3.46	69.19	10.000	8/4 (Thin overlay3/4 in)
S2	4.12	82.48	5.000	7/4 (Thin overlay 3/4 in)
S3	3.19	63.79	8.000	8/8 [Heat/planer (+rejuv./mat.)]

6. PRIORITIZING EQUATION FOR INTERVENTION WORKS

According with the prioritization equation (1) the priority index Y, is expressed as follows:

$$Y = 5.4 - (0.0263X_1) - (0.0132X_2) - [0.4 \log(X_3)] + (0.749X_4) + (1.66X_5), \qquad (1)$$

Where:
Y = the priority index ranging from 1 to 10, with 1 representing very poor, and 10 representing excellent technical condition.. Thus a lower value indicates a pavement has a higher priority for treatment.
X1 = average rainfall (in./yr);
X2 = freeze and thaw (cycle/yr);
X3 = traffic (AADT);
X4 = Present Serviceability Index;/PSI;
X5 = distress (a subjective number between -1 and +1).
In Table 5 from below, the synthetic results, obtained on these three sectors, by using the prioritization equation, are given:

Table 5. Results obtained using equation prioritizing

Sector	X_1	X_2	X_3	X_4	X_5	F
S1	20	10	10.000	3.46	0.46	6.49
S2	20	10	5.000	4.12	0.55	7.26
S3	20	10	8.000	3.19	0.43	6.28

A comparative synthesis of the resuts obtained by using the two investigation methods is presented in Table 6.

Table 6. Comparative table used for prioritization of road using the two methods

PRAS					EQUATION PRIORITIZING							Obs. on the degree of correlations
Sector	PSI	PCI	ADT	No. strategies	Sector	X_1	X_2	X_3	X_4	X_5	F	
S3	3.19	63.79	8.000	8/8 (Heat/planer (+rejuv./mat.)	S3	20	10	8.000	3.19	0.43	6.28	YES
S1	3.46	69.19	10.000	8/4 (Thin overlay3/4 in)	S1	20	10	10.000	3.46	0.46	6.49	YES
S2	4.12	82.48	5.000	7/4 (Thin overlay 3/4 in)	S2	20	10	5.000	4.12	0.55	7.26	YES

7. CONCLUSIONS

In acordance with the results presented in the comparative Table 6 and based on the existing evident correlation, on may conclude that both methods can be used in practice, function of the available investigation facilities and available time.

References

1. Hugo V, PRAS/ Dissertation Thesis, Marryland University, USA,
2. http://www.ce.tuiasi.ro/~randrei/

Evaluation of CBA versus LCA Methodological Approach for Road Infrastructure

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Summary

The features and interdependences between environment and economics, connected to the transport infrastructure sector, can be evaluated by a wide diversity of decision – support tools. The characteristics of these tools and the gaps associated with the informative system can lead to misunderstandings and misleads. The present paper aims, along a comparative analysis, to evaluate the characteristics, the benefits and limitations of two decisions – support tools, CBA and LCA. The objective is to highlight the methodologies common features and conclude on their usage and viability. The evaluation was performed along the context of environmental impact assessment in road infrastructure sector. Performing this study was intended to facilitate the users' analytical choice and to offer a personal overview on the field.

KEYWORDS: CBA versus LCA, comparative analysis, environmental impact, road infrastructure

1. INTRODUCTION

There is a wide diversity of methodologies that have the purpose to evaluate the environmental impact associated with different products or procedures. The approach is reaching more towards the economic consequences of this ecologic impact. The instruments that approach this area have common features but also differences in their evaluation. The diversity of these approaches can limit the results credibility. Here are some instruments of analysis: Evaluation of the Ecological Impact (EIA), Life Cycle Assessment (LCA), Environmental Risk Assessment (ERA), Cost Effectiveness Analysis (CEA), Cost Benefit Analysis (CBA). This list can be extended to some other examples, but the previously mentioned ones are being considered the most relevant.
For the present paper the interest has been focused on two of these methodologies namely Life Cycle Assessment (LCA) and Cost Benefit Analysis (CBA),

highlighting on the consequences of different actions associated with the implementation of a certain initiative.
The different analitical methodologies have the purpose to create scientifically based decision making information for a sustainable society. In order to sistematize the analysis, both of these methods are modelling the causality chain that incorporates a succesion of phases, shown in Figure 1 below.

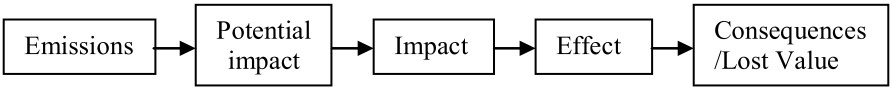

Figure 1. Causality chain for CBA and LCA

Considering the sequential evaluation, these methods can be associated to different product or process systems. For the particular case of the present paper, the evaluation makes refference to the transport infrastructure sector. As a general observation, the main utility of these methodologies can be observed in the case of comparing two or more project alternatives. The information they provide is extremely useful in establishing the economic and ecologic consequences of a certain project proposal and offer a support for decision taking.
Figure 2, presents the paper approach on the topic, along the comparison betwenn the two methodologies.

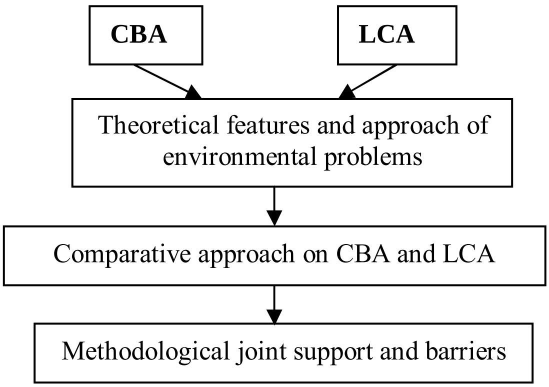

Figure 2. Comparative methodological approach

2. COST BENEFIT ANALYSIS (CBA) FEATURES AND EVALUATION

CBA is a methodology that identifies the projects and policies that have the potential to maximize the benefits of the society by evaluating all the costs and benefits. It's theoretical background sends back to 1960s and correlates to the

theory of economic wealth developed in USA. Currently is considered as a support instrument for ecologic regulations, being widely accepted and used in practice.
The wealth theory, the fundament of CBA, deals with finding the optimum allocation of the resources in the society.
As a conclusion, CBA can be considered as a prioritization instrument for investment projects and as a method for evaluation of the costs and socio – economical benefits associated to different scientific area.
At international level, currently, there aren’t detailed standards, associated to CBA methodology. The only technical documents are some general guides.
Moller et al. (2000) see the CBA approach as project/problem description, consumption description, monetary evaluation, discounting, economic evaluation of general wealth, incertitude evaluation.
The Danish Ministry of Finance has a relative similar view considering identifying the objectives, the alternative presentation, monetary identification, quantifying and evaluating the costs and benefits for each alternative and final information evaluation.
Even though the general approach on the topic is similar, several inconsistencies of the approaches can still be mentioned.

2.1. General approach of CBA methodology

In order to perform a CBA and create the basis for a proper evaluation of the impacts it is necessary to establish the project and associated problems.

1. The problem formulation:
   - ✓ Description of the context before the project implementation,
   - ✓ Evaluation of the change in activities, if the project represents a new activity or replaces an old one,
   - ✓ Resources relocation,
   - ✓ Project role.

2. Identifying and quantifying the project ecologic and economic impact:
   - ✓ Direct (due to a project implementation) and indirect (from a project direct effects) consequences,
   - ✓ Establishing the utilities and environmental impact of a project,
   - ✓ Associated adding, eliminations or replacements,
   - ✓ Externalities quantification,
   - ✓ System delimitation:
     - ➢ the economic and ecologic impact external to the geographic sphere of the project aren’t included in the evaluation,
     - ➢ the service life of the project considering the time period when it is considered to have economic and ecologic consequences.

2.2. Potential and limits of CBA

CBA is useful for project evaluation by analyzing the impacts associated to a project alternative, associating a monetary perspective to different types of impacts. This way is being created a base for comparison and a prioritization for project alternatives.
Independent from the positive components offered by CBA, must be overviewed also its limitations. These refer mainly to the discrepancy between the theoretical and practical level. The theoretical level includes methodological problems, attached to applying CBA, freely from the method development. The practical level implies that the problems are related on how CBA is being used or not in practice. Before deciding on accepting or rejecting CBA, must be considered the problems at theoretical level in all the execution phases.

2.3. Environmental approach of CBA

The CBA analysis excludes some components of the ecologic impact related to a project, even more due to the fact that a part of the ecologic impact is being reflected outside the national geographic border.
In case of comparing two environmental projects, will be preferred the one with the majority of the pollution activity outside the national borders.
According to CBA only a limited part of the impact on people should be considered. This implies that a large part of the pollution is being left outside the evaluation. Even more, only primary environmental impacts are being included in CBA, overlooking other risky impacts.
Some features specific to CBA refer to:

- ✓ The financial evaluation of costs and benefits is reduced to impacts easily evaluated or which have been previously evaluated. This way important benefits are being ignored because is difficult to associate them monetary values,
- ✓ CBA didn't developed a way to evaluate irreversible changes,
- ✓ The uncertainties can determine deformations in CBA results, leading to wrong approaches of the proposed objectives for a proposed project,

3. LIFE CYCLE ASSESSMENT (LCA) CHARACTERISTICS

LCA methodology evaluates the potential environmental impacts and the usage of natural resources along the service life of a product or process. The main feature specific to LCA is the evaluative process of product service life considering a series of consecutive steps, from material production process until final disposal.
Namely these phases refer to:

1. Material extraction,

2. Production,
3. Usage,
4. Disposal/Recycling of the product.

Along each of these phases, results a certain environmental impact due to materials exploitation, energy use, chemical processes etc. Due to these consequences is useful the usage of LCA methodology. It can compare different initiatives and product solutions based on their potential impact on the environment and decide on the most sustainable alternative. The purpose of LCA is to create eco-friendly products and services during the service life.

LCA dates back in 1990, when SETAC developed this methodology by publishing the "Code of Practice", followed by a list of ISO standards, 14040 – 3, in order to provide more credibility.

The evaluation along life cycle approaches has been extended with other components like Life cycle costs (LCC) or Life cycle management (LCM). Complementary, in order to facilitate the complex evaluation of LCA, has been created a wide diversity of calculation instruments with extended databases.

LCA methodology approaches the evaluation in two ways:

- A descriptive perspective consisting of development of product systems, eco-labeling, marketing and evaluations of potential environmental impact associated with a product/service,
- A comparative perspective due to comparisons between different project alternatives.

LCA has an unique structure due to the "cradle to" approach and orientation on products or services service like.

LCA is being perceived as a complex analysis instrument, difficult to apply in practice mostly due to the important quantity of data necessary to perform the analysis.

4. COMPARATIVE PERSPECTIVE ON CBA AND LCA METHODOLOGIES

CBA and LCA methodologies have the objective to evaluate the economic and environmental consequences associated with a project proposal, in order to change, if necessary, the current policy and offer a viable alternative.

4.1. Comparison of CBA and LCA

Observed jointly, CBA and LCA methodologies have similarities but also a lot of discrepancies. Even though these might prove to be inconsistent sometimes they can make the difference.

Considering the two instruments at theoretical level, we can start with the fact that CBA and LCA are based on different schools of thinking. LCA is based on natural sciences and potential environmental impact, while CBA originates in economic wealth theory, correlated with individual preferences but also social and ecologic features. This explains the LCA analysis orientation towards the environmental component more than CBA.
In a more detailed perspective, LCA objective is to reduce the negative impact on environment associated to a certain product, project or policy, while identifying the main differences of impacts between alternatives. On the other hand, CBA objective is to maximize the total value of a society utility by quantifying the benefits of any initiative and considering economic, ecologic and social impacts.
One of the most important discrepancies between methodologies refers to the usage of data bases and associated computer software. LCA is widely associated with informatics while CBA analysis is mostly performed manually.
Table 1 below presents several features associated with the two methodologies.

Table 1. Comparative features on CBA vs LCA

	LCA	**CBA**
Objective	Reduce the negative impact on the environment	Maximize the society utility
Focus	Environmental impact	Economic and ecologic impacts
Theoretical fundaments	Natural sciences	Economic theory, natural and social sciences
Development	Practice in industry and universities. Standardized	Sustained by public agencies and universities. No standardization
Environmental approach	Mid-point	End-point
Geographic scope	Assessment of regional and global impacts, less adequate for local impacts	The global perspective is suitable for environmental problems without considering the geographical extent. More adapted for a national approach.
Time delimitation	Considers the impact period. It is more suitable for environmental problems where the time horizon can be determined.	Not suited to deal with problems with a perspective longer than 30 years.
Transparency	The data are presented in LCI	Is lost due to aggregation
Uncertainty	Less valuable the higher is the uncertainty. Unsuitable in case of ignorance.	Less valuable the higher is the uncertainty. Unsuitable in case of ignorance
Irreversibility	Applies rankings to account for irreversible changes.	Unsuited to evaluate initiatives that cause irreversible consequences
Distribution of costs and benefits	Extreme caution if costs and benefits are unequally distributed	Extreme caution if costs and benefits are unequally distributed

4.2. Complementary features of CBA and LCA

The two decision support methods, CBA and LCA, can be applied on similar topics but with different orientations. This means they can, not integrate but support each other. LCA has a limited orientation as compared with CBA, which means it can support CBA, more than the other way. CBA can use LCA to evaluate the environmental impact and identify hidden features that might influence the results. LCA can also use CBA results, which contains information on the effects of implementing a certain initiative on a system.

The main common feature of the two methods refers to the fact that they assist the decision making process for public interest.

In case of applying a CBA – LCA methodological approach for a project proposal, first should be exceeded some barriers that might limit the cooperation between these instruments. They refer to:

- Differences in the way of handling the ecological information. Some consider that CBA should be based on LCA results,
- Differences in evaluated information. Associating a monetary value to LCA, will be realized a substitution of CBA features,
- Although the systemic analysis and information on environmental features collected along CBA and LCA analysis are similar, LCA results aren't directly applicable to CBA because of differences in time approach of impacts and geographic delimitations,
- LCA shouldn't be used to replace CBA because the evaluated information is different. CBA includes more information than environmental aspects supported by LCA.

5. CONCLUSIONS

Performing a comparative evaluation on two systemic analysis methodologies, Cost Benefit Analysis (CBA) and Life Cycle Assessment (LCA) has brought into light a diversity of common and different features. Even though the instrument can be considered as complementary due to their availability to evaluate similar problems and support similar decision, they can't be replaced. Still, in case they are being used as support tools for each other, they might become useful and stronger in the process of decision taking.

Even though the paper describes in a detailed manner the two methodologies it can be concluded on the main features of each.

CBA is based on wealth theory implying the existence of a perfect market, which is a utopia, due to the presence of public goods, externalities, increase of market reimbursement and asymmetric information. This instrument considers that all the consequences must base on monetary values and rejects the idea that not everything is being expressed in monetary terms.

CBA can exclude from the analysis the environmental impacts, due to the constraints in monetary evaluation and system delimitation it considers the risks issue in a simplified manner, overlooking the potential catastrophic effects and irreversibility. It is a decision taking tool, easy to handle.

The more recently developed methodology of LCA is a decision support tool, more complex, supported by databases and computer software. Some consider this instrument as too complex, time consuming and difficult to perform. Still it gains credibility due to the systemic approach of problem solutions, to its standardization efficiency and extended usage in the sector of product development and marketing for private companies.

LCA is the only instrument used to compare the environmental impacts associated to products, processes and policies, is very useful and can't be replaced by other instruments.

Observed jointly, the two analytical instruments are very similar, still important limitations and differences maintain. Their most valuable characteristic is the possibility to associate to products and projects from all areas of activity, helping along the process of choosing the most viable alternative.

References

1. Hansen M. S., Gilberg U.M., Master Thesis: Cost-Benefit Analysis and Life Cycle Assessment in Environmental Decision – Making – Potentials and Limitations, Technical University of Denmark, 2003, publication no. IPL – 172-03.
2. Dreyer L.C., Niemann, A.L., Hauschild, M. Z., Comparison of Three Different LCIA Methods: EDIP97, CML2001 and Eco-indicator 99, The International Journal of Life Cycle Assessment; Volume 8; Number 4, Ecomed publishers, 2003.
3. Finnveden G., ”Valuation Methods within LCA – Where are the Values?”, The International Journal of Life Cycle Assessment, Volume 2; Number 3; pp. 163-169, Ecomed publishers, 1999.
4. Gerrod, G. & Willis, K. G., Economic valuation of the environment, methods and case studies, Edward Elgar, England, 1999.
5. Hanley, N.; Spash, C. L., "Cost-benefit Analysis and the Environment", Published by Edward Elgar Publishing Limited; Great Britain, 1993.

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Environmental Impact Assessment of Flexible Asphalt Pavements Integrating RAP

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Summary

The assessment of environmental impact associated with the production, laying and compaction the asphalt pavement is a complex frame of factors, actions and consequences. Currently, the tendency towards sustainable actions embanked a frenetic search for innovative solutions.
The present paper carries out an evaluation of the environmental impact associated with road flexible asphalt pavement integrating Reclaimed Asphalt Pavement (RAP) in difference percentages. The assessment is expressed as amount of equivalent CO_2 emissions per ton of mixture.
In order to obtain reliable results, the analysis has been performed using LCA methodology in a ”cradle to site” perspective.

KEYWORDS: environmental impact, flexible asphalt pavements, Reclaimed Asphalt Pavement (RAP), LCA

1. INTRODUCTION

The evaluation of environmental impact associated to a certain product, service or procedure is highly important for the determination of the process sustainability. As regarding the impact resulted from a road construction process, the analysis exceeds the material and economic features towards a more complex evaluation. A fundamental aspect in understanding and evaluating the environmental effects implies obtaining information on materials, products and processes, comparing the alternatives and choosing the right strategy in order to reduce the CO_2 emissions.
The environmental impact of transport infrastructure construction works can be determined by computing the value of kg CO_2e emissions during the pavement life cycle. This analysis can be facilitated using LCA methodology, which allows a gradual evaluation of the consequences due to the “cradle to” perspective [7].
The present paper objective was to identify the most feasible road pavement construction alternative, from environmental point of view, for a particular example. In order to perform this analysis has been considered the LCA methodology and incorporated into asPECT [2] computer software.

The process consisted in comparing the road construction process using asphalt pavement bitouminous mixtures based on Reclaimed Asphalt Pavement (RAP) or “virgin” materials. It consisted in evaluating the environmental impact as kg CO_2e emissions in a “cradle to site” perspective using asPECT software.
Using LCA methodology implies evaluating all the phases starting with the extraction of raw materials untill asphalt mixture laying and compacting [2].
This type of evaluation helps the road construction companies to identify and select the materials and processes with the lowest energy and water consumption, with minimal CO_2e emissions during the product’s life cycle, minimal costs and traffic disturbance [3].

2. LCA METHODOLOGY AND ASSOCIATED COMPUTER SOFTWARE

LCA methodology can be used to quantify the environmental impact of a product or process over its life cycle, starting with raw materials extraction until final disposal [8], which can be envisaged in Figure 1.

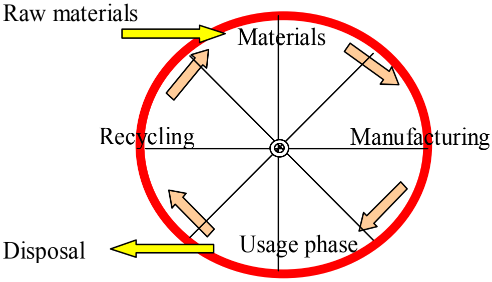

Figure 1. The product life cycle

In the present research, based on a comparative process, this methodology used to evaluated among different production processes and find the most eco-friendly alternative.
According to the “cradle to site” perspective and the product service life, the LCA methodology was used to evaluate the environmental impact associated to a road construction process.
The construction process can be considered as dynamic, adaptable and opened for innovations, beyond traditional procedures.
In the last years, researchers and institutes have developed computer software that associated to the LCA methodology facilitate and optimize the evaluation of environmental impacts of construction works [11].
One of these software, which has also been used to perform the analysis in the present paper is the Transport Research Laboratory (TRL) product, named

asPECT, specialised on performing LCA analysis for asphalt pavements. It provides a framework which contains the necessary formulae, emission factors and database for the evaluation of CO_2e emissions of asphalt products [2].

3. RECLAIMED ASPHALT PAVEMENT (RAP) ACCORDING TO LCA PERSPECTIVE

Currently, the process of recycling the used materials is of high interest. Extracting the benefits resulted from their recyclability means assessment and comparison of different product systems.

LCA is the methodology provides extended facilities along the evaluation process of material recycling [5]. The analysis results lead to the creation of alternative scenarios for a product service life. The objective is to design products with high recyclability potential, in order to reduce the future ecologic burden.

The transportation agencies in cooperation with researchers, are working out to find a way to best approach the problem of environmental impact and economic feature involvement.

The recyclability process in the sector of asphalt pavements gain interest since the 1970 campaign of recycling the old asphalt material, known as RAP [1].

The RAP can be recycled and incorporated in different types of mixes like Hot Mix Asphalt (HMA), cold mixes or as aggregate in granular or stabilized base materials [4] next to "virgin" materials. The technological development in plants and equipments was the key element that made possible the increase of the RAP percentage in asphalt mixtures up to 50% or more.

Currently, it is the most reused material in the world, with values nearly twice as much as the combined total of recycled paper, glass, aluminium and plastic. USA, is the country that mostly uses RAP into asphalt mixtures, followed by many organisations and associations, all around the world [4].

The present paper focused on the environmental impact associated to HMA including RAP. The asphalt pavement recyclability starts with the milling process. The extracted materials can be either recycled, resulting RAP, or disposed in a landfill. The process gains support due to the development in the equipments and techniques. The most important feature in producing asphalt mixtures incorporating RAP is ensuring an adequate mixing of new and RAP materials, that complies with the technical and quality performances of the product [1].

Currently, all national transportation agencies allow the usage of RAP, in a minimum percentage of 20% - 25%. Still, this percentage can reach values of even 70-80% in USA [4].

The restrictions in the usage of RAP mainly relate to durability problems associated to the recycled materials. In order to obtain a maximum exploitation of RAP benefits, have been implemented studies in order to find components, consisting of materials or additives that might increase the quality and performance of asphalt pavements using RAP. An example of additional components is additives like Sasobit® and Advera®zeolite [9] that provide increased workability, higher moduli, and strengths. The economic analysis suggests important savings resulted from higher usage of RAP percentages [9]. The environmental analysis consists of reducing the kg CO_2 equivalent emissions resulted from the production of asphalt pavements new or recycled.

4. THE ENVIRONMENTAL IMPACT OF NEW AND RECYCLED ASPHALT MIXTURES

Along the service life of a road, appear failures that sometimes might find the maintenance works turn useless. At that point must be taken into consideration the replacement of the asphalt layers. The technical procedure implies milling the asphalt at different depths, depending on the failures [3].
For the present paper was considered the evaluation of the environmental impact associated with the construction of a road asphalt pavement using either traditional or recycled materials.
The case study implied a road section with the dimensions of 1000 m length, 7 m section width, 4 cm thickness, total tonnage 280. The analysis process implied laying asphalt mixtures using local traditional compositions: BA 16 (asphalt concrete that embeds aggregates having a maximum diameter of 16 mm), BAD 25 (open grade asphalt concrete that embeds aggregates with a maximum diameter of 25 mm) and AB 2 (bituminous "anrobat" with coarser aggregates), detailed in table 2.

Table 1. Traditional asphalt mixture composition

No.	***BA 16***		***BAD 25***		***AB 2***	
	Material	***%***	***Material***	***%***	***Material***	***%***
1	Bitumen	6	Bitumen	4.5	Bitumen	4.2
2	Chippings 8-16	24.4	Chippings 16-25	23.9	Crushed gravel 16-25	17.2
3	Chippings 4-8	14.1	Chippings 8-16	23.9	Crushed gravel 8-16	14.4
4	Crushed sand	45.1	Chippings 4-8	9.4	Crushed gravel 4-8	19.2
5	Filler	10.4	Natural sand	17.2	Natural sand	19.2
6	-	-	Crushed sand	16.3	Crushed sand	19.2
7	-	-	Filler	4.8	Filler	6.6

In order to facilitate the evaluation, a comparison has been performed only for the first upper layer, BA16, between traditional mixture and one that incorporates RAP, named BA^r16. The recycled mixture incorporates a value of 75% RAP and a low percentage of bitumen (2%) as detailed in table 2.

Table 2. Recycled asphalt mixture composition

No.	*BA^r 16*	
	Material	***%***
1	BA 16	75
2	Bitumen	2
3	Chippings 4-8	10
4	Crushed sand	10
5	Filler	3

The analysis consisted of characterizing two road asphalt mixture alternatives from environmental impact point of view, expressed as kg CO_2e emissions/ton of asphalt mixture [8]. In order to facilitate the analysis and give viability to the results, have been used LCA methodology and asPECT computer software, performed in a “cradle-to-site” scenario. Table 3 shows the analysis results, divided for traditional and recycled asphalt mixture.

The analysis results showed a reduction of the environmental impact of 38%, expressed in kg CO_2e emissions for the recycled mixture as compared with the traditional one. Important decreases of the impact can be extracted along the transportation phase, depending on transportation alternatives used, types of energy source or equipments for in-situ construction works.

Table 3. Environmental impact assessment [8]

Material		Layer depth (cm)	Quantity of mixture (t)	Total CO_2e		Total kg CO_2e /t	
				Traditional asphalt mixture	Recycled asphalt mixture	Traditional asphalt mixture	Recycled asphalt mixture
BA 16	BA^{r}16	4	644	**125.437,7**	**92.592,2**	**194,8**	**143,8**

This type of comparison can be developed for any type of constructions alternatives, considering the quantities, transport distances, asphalt treatments or additional lifetime used.

5. CONCLUSIONS

The assessment of the environmental impact, associated with the construction processes, represents a feature of high importance for the construction sector [8]. Using LCA methodology and asPECT computer software can be identified the most environmentally friendly construction alternatives. These complex instruments evaluate a vide diversity of environmental effects associated with a given road structure, its service life, reuse alternatives and demolition procedures.

The present paper promotes the asphalt recyclability process, acquisition of new

and improved equipments, use of the LCA and computer software.
The main conclusion of the research was that, in road projects, replacing the traditional asphalt mixture with recycled ones, can obtain a significant reduction in environmental impact. The analysis results (a decrease of 20% to 40% of environmental impact) promote the usage of RAP into asphalt mixtures and the benefits generated from the process.

References

1. Al-Qadi I., Elseifi M., Carpenter S., Reclaimed Asphalt Pavement – A Literature Review, Research Report FHWA-ICT-07-001, Illinois Center for Transportation, Illinois, USA, 2007.
2. asPECT Protocol and Guidance Book, (2009), Transport Research Laboratory (TRL), United Kingdom, 2009.
3. EcoLanes (Economical and Sustainable Pavement Infrastructure for Surface Transport) 031530, D4.1 a, Environmental Impact and Energy Consumption of Transport Pavements, FP6 STREP Project, 2007.
4. FHWA (Federal Highway Administration), Asphalt Pavement Recycling with Reclaimed Asphalt Pavement (RAP), US Department of Transportation, 2011.
5. Five Winds International, Critical Review of the Life Cycle Assessment (LCA), 2007, http://www.pe-international.com/fileadmin/ user_upload/news/2007 /downloads/Critical_review_02.pdf.
6. Jugo A., Development of a Rehabilitation Methodology Approach Hierarchy for Flexible Highway Pavements, PhD Thesis, Faculty of Graduate School of the University of Maryland, USA, 1986.
7. Mora P.E., Life Cycle, Sustainability and Transcendent Quality of Building Materials, Building and Environment, p.1329-1334, 2007.
8. Nicuță A.M., Life Cycle Assessment Study for New and Recycled Asphalt Pavements, The Bulletin of Polytechnic Institute from Iasi, Constructions, Architecture, LVII, 81-91, 2011.
9. O'Sullivan, K. A., Wall, P. A., The Effects of Warm Mix Asphalt Additives on Recycled Asphalt Pavement, Worcester Polytechnic Institute, Worcester, 2009.
10. West R., Reclaimed Asphalt Pavement Management: Best Practices, National Center for Asphalt Technology, NCAT Report, Auburn University, Auburn, 2010.
11. Zhang Z., Wu X., Yang X., Zhu Y., BEPAS – a Life Cycle Building Environmental Performance Assessment Model, Building and Environment, p. 669-675, 2006.

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Stone bridges from period 1450-1650, to the geographic and historic area of Romania

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Summary

Lucrarea reprezintă un studiu istoric privind podurile de piatră construite, în special, în provincial istorică Moldova, în perioada 1450-1650. Studiul se bazeză pe analiza unor documente consacrate din domeniul Istoriei şi pe cercetări de teren, ocazie cu care s-au identificat podurile menționate în articol şi aflate unele încă în exploatare. Podurile descriese în lucrare sunt construite în Moldova condusă de doi importanți domnitori: Ştefan Cel Mare, care a domnit între 1457 şi1504 şi Vasile Lupu (1634-1655).

Podul de la Borzeşti este construit pe drumul Adjud – Târgu Ocna – Oituz, peste pârâul Ştefan Cel Mare. Podul este realizat din piată (bolovani de râu) cu mortar de var gras, cu rosturile înclinate pe intrados şi având bolovanii din față cioplíți.

Podul a fost reparat, de inginerul George Sion, între 1902 – 1903, prin realizarea a două diguri de apărare, înlocuirea bolovanilor căzuți din boltă, care a fost, mai apoi, tencuită cu mortar de ciment. În prezent podul nu mai este în exploatare, dar se găseşte într-o stare tehnică bună.

Podul de la Cîrjoaia este construit, peste pârâul Cirjoaia, din blocuri de piatră cioplită, sub forma unei bolți eliptice cu o deschidere de 7 m. În anul 1847 podul a fost restaurant pe timpul domnitorului Mihail Grigore Sturdza .

La Zlodica, sat component al Cotnarilor, pe drumul comunal DC 156 există, de asemenea, un podeț construit masiv din piatră cioplită, care este menționat într-un document din 1806 aflat la Arhivele Statului din Iaşi.

Pe drumul național numărul 15D, Roman – Vaslui, la km. 116 + 500, în localitatea Cănțălăreşti, comuna Ştefan cel Mare, a funcționat până 1986 un pod de piatră, peste pârâul Racovățului. Structura de rezistență este o boltă în plin cintru cu o deschidere de 3,20 m şi a fost executată din zidărie de piatră cioplită, între anii 1635 – 1636, de către hatmanul Gavril Goci, fratele domnitorului Vasile Lupu. Are o grosime la cheie de 3,80 m şi o lungime totală de 21,25 m şi asigura o lățime de carosabil de 5,50 m.

KEYWORDS: *pod de piatră, boltă, pod istoric.*

1. THE BRIDGE OF STEFAN THE GREAT

Since Trajan's time (98 – 117AD) until Stefan The Great (1457 – 1504 AD) there are no proofs, neither in writen documents nor in archaelogical discoveries, that arched stone bridges were built, bridges with a permanent character similar to those built by the Romans.

The only evidences, arches over the time, like a hand stretched by the ancestors of today's and tomorrow's generations, are the arched stone bridges, Roman type arches in full semicircular arch. On the national road No 11A, at Negoesti, at the entrance in Stefan Cel Mare commune, on the right side, on the other part of the railway embankment, the remains of the old national road which would link the cities of Bacau and Onesti can still be seen. There is also another fully arched bridge, over a creek having a torrential character and a deep, marked valley, which atenuates its fury in the Trotus river; the vault has a 5.80 m span, is made of river boulders in two rings, bound with thick lime mortar, and was built during Stefan Cel Mare's reign (1457 – 1504). This bridge is presently known as the Bridge of Borzesti (the birth place of Stefan The Great, which is not too far from the location of the bridge).

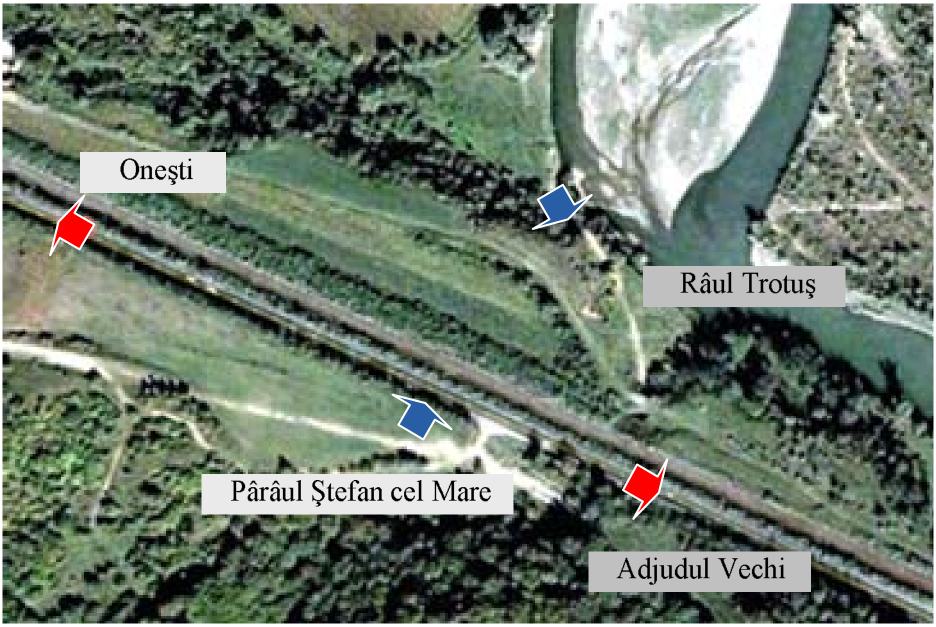

Figure 1. Satellite image. Location of Stefan The Great Bridge. A project classified as a historic monument, built during the Voivode's Reign, 1447-1504

Figure 2. Borders of Moldova during Stefan The Great's reign

The historic context of such a project is marked by a period in which the whole defensive system of Moldova was consolidated and completed:

- Fortress at Roman city was rebuilt with stone masonry
- Suceava Fortress was successively transformed reaching the status of fortress-palace.
- To Neamt Fortress: new fortification belts were added
- In 1479 Chilia Fortress was built

Under Stefan The Great the country of Moldova undergoes a period of economic development, and blooming of commerce. Moldova was exporting goods to Poland, Turkey and Transylvania goods like wheat, oats, barley, rye, cattle, hogs, horses, fox and squarell pelts, fish, honey, wine etc). Moldova would import silk, incense, cinnamon, peper and Greek wine. All these made the movement of goods to determine the development, to the extent possible, of the transportation infrastructure.

Figure 3. Commemoration monument from ruler Mihail Gr Sturdza's reign.

The roads generated by the increasing needs of trade were of great importance for the economic life of Moldova but, to the same extent they were important for the nieghboring countries. The most important roads of the country of Moldova, evidenced by historic documents, were:

1. Iasi – Vaslui – Barlad – Tecuci – Galati;
2. Focsani – Adjud – Bacau – Roman – Targu Frumos – Botosani – Mihaileni, with branches to:
    a. Targu Frumos – Iasi,
    b. Adjud – Targu Ocna – Oituz,
    c. Bacau – Piatra – Prisecani,
    d. Roman – Falticeni – Suceava;
3. Iasi – Dorohoi – Mamornita – Cernauti;
4. Iasi – Husi – Foltesti – Galati.

Figure 4.The elevation of the arched bridge built during Stefan The Great's reign (1457 – 1504)

The first record of the arched bridge constructed during the ruler Stefan The Great's reign is found in the specialized literature of Romanian academician C. I. Istrate in 1903, who mentioned this bridge under the name of Borzesti Bridge. Actually the bridge is located on the territory of Negoiesti Commune, on the road Adjud – Targu Ocna – Oituz, over the creek Stefan The Great (or, in Romanian, Stefan Cel Mare,). In the writings of academician C. I. Istrate it is mentioned that during the ruler Mihail Dimitrie Sturdza's reign (1834 – 1849) a small commemorative memorial was raised to "remembrance of bridge construction"; according to our traditions the memorial was altered on the occasion of the construction of Adjud – Onesti railway (1881 – 1885).

In 1873 the upgrading of the county road Adjud – Onesti – eastern border started. Upon its completion it was classified as national road. As engineer George Sion is telling us, I quote: *„among the bridges existing at that time, at milestone 24+810 (measurement starting from Adjud) an arched bridge made of stone was found, built during Stefan The Great's reign; the river it crossed was called Stefan The Great's Creek. The bridge is built from stone blocks with fat mortar, the vault bing made of round boulders, and the arch's ends, which can still be seen today, are also made from rounded boulders. The road was placed were the first cornice was, at 5m30cm above the bottom slab, having as parapet the height between the two cornices of 1m 20cm, but in time ...".*

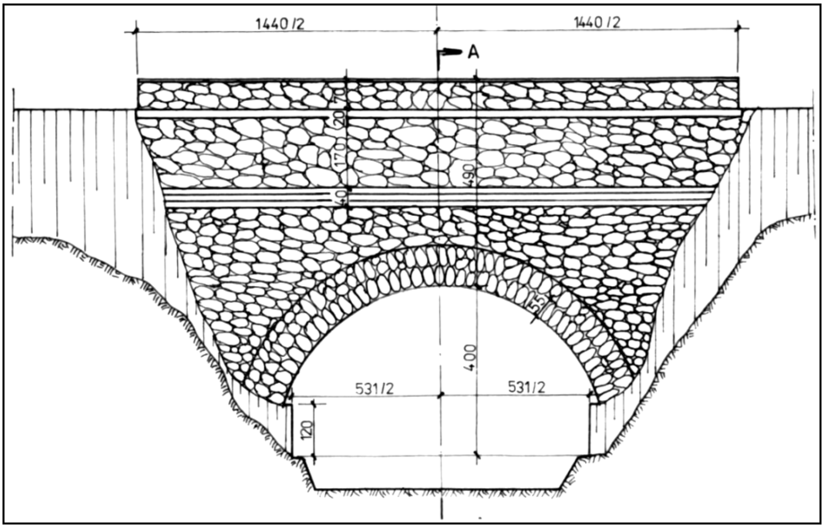

Figure 5. Reconstitution of the arched bridge, done by Liviu Popovici in 2009

In the year 1902 – 1903 several repair works were done on this bridge under the management of engineer George Sion, concomitantly with some river bed consolidation works consisting of bottom thresholds. After 1930 the national road No 11A has been superelevated, which can be observed in the picture, the line of tympanums being shown distinctly between old and new.

Figure 6. Blamable consquent actions of those who take, as souvenirs, stones from the arch's base.

Following the construction of a local by-pass road, presently the bridge is not anymore in operation; it is being used only by the local people on their way taking the cattle to grazing. A commemorative plaque dating back to ruler Mihail Sturdza’s reign (1834 – 1849) dissapeared long ago. Today the place is signaled by a milestone and abandoned.

Many visitors have snatched and continue to snatch pieces from the arch’s intrados base, as souvenirs. A blamable act for such an important historic monument.

CANTALARESTI BRIDGE

On the national road No 15D, Roman – Vaslui, at milestone 116+500 km, at Cantalaresti, the commune of ruler Stefan The Great, a semicircular arch, over the creek Racovat, having a span of 3m20cm, built between 1635 – 1636 from stone masonry, has been in operation until recently; it was built by hetman Gavril Goci, brother of ruler Vasile Lupu (1634 – 1654).

Figure 7 Refurbishment of Cantalaresti Bridge done in 1986. Photo by Loredana

Cantalaresti Bridge is 3m80cm thick at keystone and has a total length of 21.25 meters, ensuring a 5.50m carriageway. In an ordinance of 1686 there are data about the refurbishment of this bridge.

Being declared a historical monument, a local detour road and the performance of some conservation works, including the development of a water spring (which quenches the thirst of tourists) and a parking space, has put the bridge out of its current operation, its function as a bridge being preserved only for those who have a little time to spare for stopping by an meditate.

Figure 8. National Road DN15D, Roman – Vaslui, km116+500. The old medieval bridge at Cantalaresti. Photo by Loredana.

For us, Romanians, this bridge has a particular meaning. Let the great historian Nicolae Iorga, with his „History of Stefan The Great", to highlight a historic moment: the battle against the Turks known under the name of „The battle of the Tall Bridge" (1475), or the Battle of Vaslui.

„In the place where Racovatul Creek flowed into Barlad River, making a swamp between the two forests, the passage was particularly dangerous. A wooden bridge would link the two ravines, a good bridge for the everyday travelers, even for the goods wagons of tradesmen, but entirely inadequate to support the weight of camels and canons, of the thousnds of horses, of tens of thousnds of people – the heap, always crowded, of the turks".

Figure 9. The old medieval bridge at Cantalaresti. Epigraphic inscription on the downstream elevation of the bridge. Photo by Loredana

„Those who managed to reach the place, climbed up the bridge deck planks, hurrying towards the death which was waiting for them in the mist. The gunners were aiming by the noise, from both sides of the main road, and the bridge colapsed with the yells of fear and pain which were soon braised by the shout which shaked the woods”

It is the victory of christianity over the Otoman expansion. The Moldavian Army (roughly 40,000 people of which 31,200 Moldavians, 5,000 Secklers, 1,800 Hungarians, 2,000 Polish), having twenty cannons, under the command of Stefan The Great, has recorded a brilliant victory against the Otoman army, which numbered 120,000 soldiers and a huge artilery, just by sacrificing a bridge. The epigraphic inscription on the downstream elevation of the bridge (the western side), or at least what is still kept from it (2010) tells us, I quote: „This bridge was made by Hetman Gavriil and Princess Liliana, during his brother’s time, Voivode Ioan-Vasile in 7144 (or 1636)”

CARJOAIA BRIDGE

On the county road that links Ciarjoaia Commune to National Road No 28B there is still today in operation a parabolic arch with a very small flatness, made of wrought stones. Initially the bridge was built on four spans of 7.00m each, and a total length of 42.00 meters of which today there is only one span in operation. Miron Costin’s chronicle (1633 – 1691) attribute this construction to Stefan The Great.

Figure 10. Bridge in Cirjoaia Commune, near Cotnari

On the occasion of restauration works carried out in 1847, on two of the stone blocks of the guardrail the following inscription was carved:

„This bridge happily built for the remembrance of sovereign Stefan The Great was renewed in the year 1847 by order of His Highness Sovereign Voivode Mihail Grigore Sturdza, under the auspieces of the ministry of the interior, chancellor Iordache, on state's expenses"

In the year 1832, The Department of Interior Affairs resorts to Harlau Stewardship drawing their attention that the repair-consolidation should be done with great care in order not to change „its true being and antiquity". This document could be considered the first technical specification in which the way how to restore an engineering piece of work (or a historic monument) should be done.

By local initiative, in the XXth century, engineer Laurentiu Olanuta, the director of local roads in Iasi county, took the decision to re-consolidate the bridge, which currently (2010) is in a very good condition, but is out of operation.

ZLODICA BRIDGE

In the same Cotnari area, which has been permanently within the attention of waywode Stefan The Great, another small arched bridge made of stone masonry, is located on the Communal Road No DC156, within the township limits of Zlodica.

This structure is mentioned in a document of 1806, which exists in Iasi State Archives, and in which it is said, I quote: ...*"At Zlodica, over Cotnari, there is Stefan The Great's stone bridge; a big bridge decked with stone slabs at the top"*

Figure 11. Bridge in Zlodica Village

Figure 12. Bridge in Zlotica Village and the location of inscription

The bridge would have on the downstream elevation a plaque on which the text mentioned above was inscripted. Plugging of the river is so heavy that the elevation is partially buried, while the other spans are totally covered with earth.

History documents mention many other structures which cannot be found today in the field. I think it is useful to mention them. Who knows. By accident or by archaelogical researches they will be discovered one day:

- 2 August 1400, the bridge of Dragomir The Forester, over the Black Creek (Paraul Negru), at Trifesti (Neamt);
- 1420, the Bridge of Garlanici over Bacovat River;
- 18 April 1444 and 12 March 1488, the stone bridge over the river Somuzul Mare, at Mihailesti (Mihailesti Village, Suceava);
- 13 September 1473, the bridge of Hasnis over Runcului River, at Berchesesti, Suceava;
- 15 Ocober 1488, the Bridge of Vetea over Black Creek (Paraul Negru), at Vladiceni (Neamt).

References

1. Cihodaru, C. ş.a., Documenta Romania Historica A Moldova, vol. I (1384-4886), Ed. Acad. RSR, 1975;
2. Colecția Documente, P. 104/10, Arhivele Statului Iași
3. Ionescu, C., Cu privire la „Drumul cel mare" şi podul de piatra de la Negoieşti din timpul domneie lui Ştefan Cel Mare, rev. „Drumuri şi poduri", nr. 12 (81), iunie 2004;
4. Ionescu, C., "Stone Bridges belonging to Stephan The Great Reign: Carjoaia and Zlodica", rev. „Drumuri si poduri", nr. 13 (82), iulie 2004;
5. Ionescu, I., "The Bridges", Polytechnic Society Bulletin, Vol I, Bucharest, 1931;
6. Istrate, C. I., Biserica şi podul din Borzesci precum şi o ochire relativă la bisericile zidite de Ştefan cel Mare, Analele Academiei Române, Seria II – Tomul XXVI, 1903-1904.
7. Ungureanu, Gh., Anghel Gh., Botez, C., "Chronicles of Cotnari", Scientific Publishing House, Bucharest, 1971.

Bridges of the Romanian middle ages

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Summary

The analysis included in this article presents a research concerning historical documents presenting the Romanian Middle Age. These documents include references about roads and bridges existing at that time.

The documents attest:

- *Roads were natural and generally overlay on the old roman roads;*
- *Medieval fortresses were built with stone walls and enclosed by large and deep ditches filled with water over which wooden bridges were erected. The wood superstructure laid on stone masonry infrastructure. Example: Suceava fortress, Neamt fortress, Hunedoara castle;*
- *Main routes are Câmpulungul Moldovenesc - Rodna – Bistrița; along Trotus river valley toward central Transilvania: along Oituz river valley toward Braşov; along Teleajen Buzăului, Prahovei river valey, and Bucureşti - Cîineni along Olt river valley and trough Lovişte toward Sibiu. Another road went up along Jiu river and Vîlcan gorge arriving to Hațeg;*
- *In Transilvania main towns were networked with roads. Cluj was connected with Dej and Bistrița, with Turda, Alba Iulia, Sebeş and Sibiu; Alba Iulia was connected by road with Mediaş, Sighişoara, Odorhei on one side and with Deva, Arad, Timişoara on the other side; Sibiu was connected with Făgăraş and Braşov.*
- *"Moldavians, but not only them, knew to build roads and bridge" is the affirmation of the scholar Dimitrie Cantemir in his work "Descriptio Moldaviae".*
- *Wooden bridges erected in that period totally disappeared. In the property and bordering documents the most used words were: podeț, podişor, podişcă, poduleț, vad (ford);*
- *Historical documents also attest the existence of fix overpasses but also floating bridges either by ferry or by vessels bridge, which were maintained by domain owner.*
- *The streets in towns were paved with wood and were named "bridges": the Ruler's Bridge on the Main Lane, the Great Bridge, Şerban Voda Bridge, Beilic's Bridge, Long Bridge, Old Bridge, Green Bridge in Iasi, etc.*

KEYWORDS: road, wooden bridge, fortress, Romanian Principates.

INTRODUCERE

The Romanian middle ages, which spans hundreds of years, is characterized by a closed economy. The economic life within the territory of Romania, which was situated in the way of peoples' migration, and the aggression of the great empires, has known a long period of stagnation. All the above made the interest for ways of communication inexistent, with direct consequencies regarding the legacy in terms of roads and bridges from the Romans, which dissapeared almost compltely due to lack of maintenance.

There were natural roads over the plains and along the valleys, following the alignment of old Roman roads. The wooden bridges were built upon wooden posts or trestles, which would dissapear during the first fury of waters. In most of the cases the rivers were crossed at shoals or by floating bridges which can be found even today in some places.

It is not difficult to imagine what a voayge would have meant at that time on the territory of Romania, by the wagon, or charriot at a later time, on the natural roads. The travel was the attribute of carter's competence and whistle. They would advance in zig-zag to avoid the potholes, leaving clouds of dust behind them. I presume that in rainy days many pairs of oxen were needed to tow the wagons out of mud.

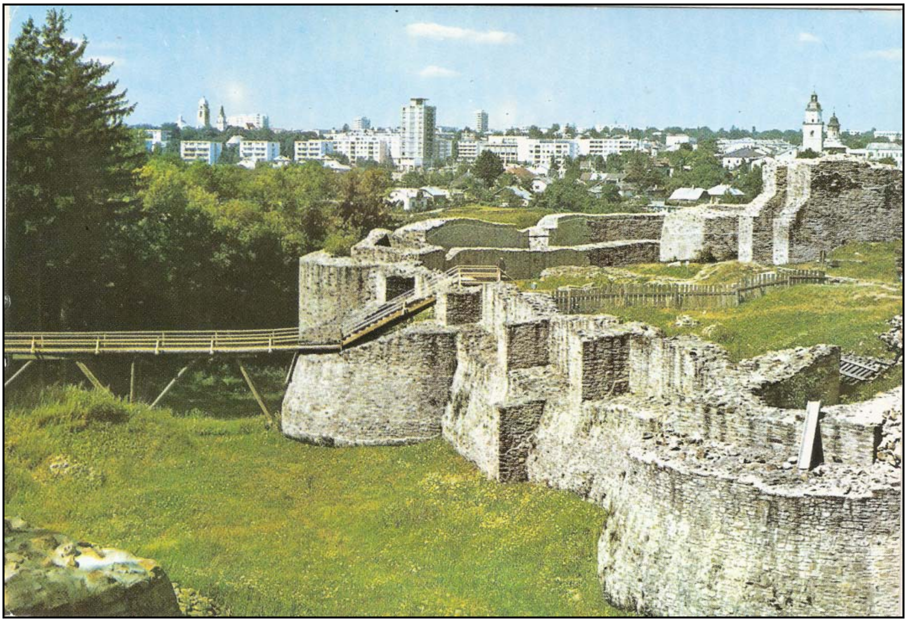

Figure 1. Suceava Fortress

PODURILE DE LEMN ALE CETĂȚILOR

The rulers of the country were those who would deal with road and bridge matters. The medieval fortresses, with thick, stone walls and surveillance towers were fortified by wide and deep ditches, filled with water, of which crossing was ensured through wooden bridges with mobile openings towards the fortress.

Figure 2. Neamt Fortress, built during Petru I Musat's reign, 1374 – 1391. The wooden superstructure was supported upon stone masonry infrastructure

Figure 3. Entrance to Neamt Fortress

For these bridges we have no data. Nevertheless, the history sources reveal that, once the first state formations occured, Muntenia and Moldova on one side of the Carpathians, and Transylavania on the other side, links between the three Romanian countries were permanent and numerous.

Over the Carpathian Mountains roads occured in several places: Campulung Moldovenesc – Rodna – Bistrita; on Trotus Valley towards the center of Transylvania; on Oituz valley towards Brasov, on Teleajen and Buzau valleys, on Prahova valley and Bucharest – Caineni, on Olt valley and through Lovistea Sibiu.

Another road would climb up the Jiu valley and, through Valcan gorges, would reach Hateg Country.

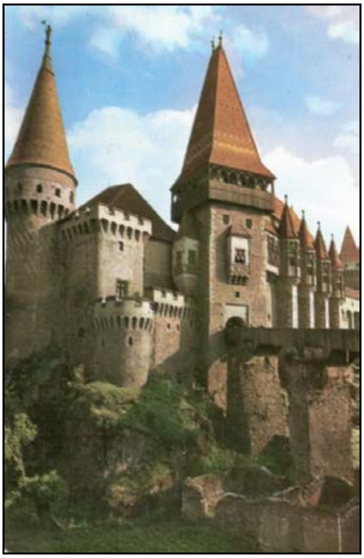

Figure 4. Hunedoara Castle, XIV – XV centuries

In Transylvania the main cities were connected by roads. The City of Cluj was linked with Dej and Bistrita, with Turda, Alba Iulia, Sebes and Sibiu was linked to Oradea; Alba Iulia was linked to Medias, Sighisoara, Odorhei, on the one hand and to Deva, Arad and Timisoara on the other hand; Sibiu was linked to Fagaras and Brasov.

“The Moldavians, but not only them, knew how to build roads and bridges”, at least this is what the well-known ruler and scholar philosopher Dimitrie Cantemir has affirmed in his masterpiece “Descriptio Moldaviae”, I quote:

“In the end, his son, Bogdan The Blind, has relinquished his Moldova to the Turks, although, it was said, according to his father’s will, under the following conditions: to grant four thousand gold coins, forty horses and twenty four hawks,

but not as a tribute but as a right to recognize the bondage, and, if the Sultan himself would take part in an war expedition, to send to the Turkish army FOUR THOUSAND MOLDAVIANS TO OPEN THE ROADS AND TO REPAIR BRIDGES".

The wooden bridges built in this period disappeared totally. The most used term in the period that we examine here, certified by bordering documents and ruler's ordinaces is:

- Culvert
- Little bridge
- Small culvert
- Small bridge
- Channel

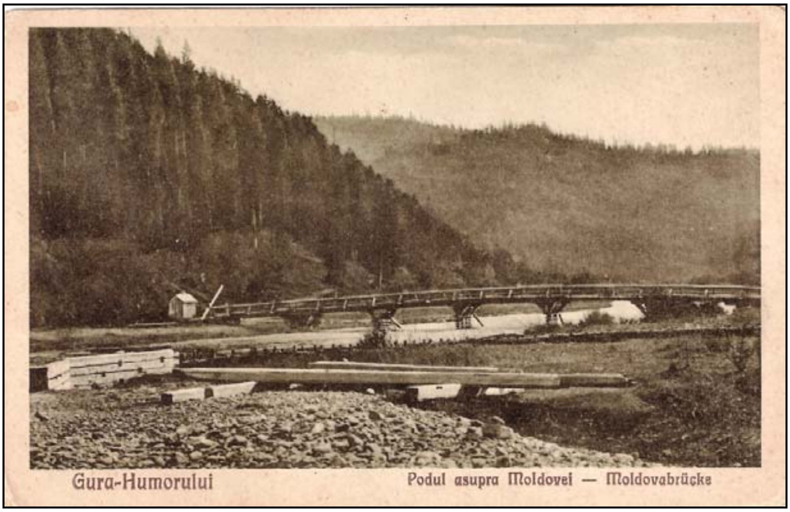

Figure 5. An wooden bridge over Moldova River at Gura Humorului,
a postcard of roughly 1930

These works were executed without any rule by which the risk of their destruction be eliminated. The smallest flashfloods would have destructive effects upon them and were taken by the water very easily. As it is mentioned in many writings, the foreign travelers were frightfully talking about crossing the rivers of Valachia and Moldova. Probably the saying "make the devil your brother until you cross the bridge", is of Romanian origin.

A lot of old Romanian villages in the Romanian territory were called valleys (Romanian, vaduri); e.g. Vadeni, Vadul lui, etc. This confirms that in those times big rivers in winter were crossed on ice and at shallow places in summer, i.e. valleys, or shoals, which were signaled along the big river banks, by big earth mounds. The history records the existence of such mounds on Siret river bank ever since the phenomenon of migration of peoples.

Figure 6. Neajlov River at Calugareni

History documents chronicled also the existence of some fixed crossings, either on bridges or floating crossings, and, rarely, on fixed bridges maintained and operated by the landowners, and by bridge entrepreneurs. Many times associations of inhabitants of a village or of several villages, called Cisla, were encountered, which were interested in maintaining and operating these works.

Figure 7. Battle of Calugareni. Painting by Theodor Palade

Tolls which were paid for crossing were known under the name of "brudine". This system generated a lot of problems due to misuses which were committed in many of those locations. There were tolls levied on those who would cross the river by swimming or on cattle and wagons which would cross at shoals, or even on those who crossed on ice in winter. In many instances the rulers had to intervene to eliminate the misuses.

From the letters of the astronomer Boscovici, XVIIth century, we learn that he crossed the river Barlad on a string bridge made from tree trunks.

Nicolae Balcescu, in his work „Romania under Voivode Mihai", which was postumuously published in 1878, tels us, I quote: *„The road that goes from Giurgiu to Bucharest crosses a flat and open plain, except in one place, at about two post spans from this capital, where it is narrowed between forested hills. Between these hills there is a valley only one quarter of mile wide, covered with bushes, which Neajlov brook floods and the creeks that flow from the hills make it a pool full of mud and bog. At that place the road goes along that valley, partly on a dirt stretch and partly on a wooden bridge, which both are so narrow that a wagon can hardly get through. This pass, which the locals call Vadul Calugarenilor, was chosen by Mihai Voivode to serve as Thermopilae to Romanians".*

It is Nicolae Balcescu too, as professor Ion Ionescu affirms in his work „Our Bridges", published in the magazine „NATURA", who accounts that the ruler Serban Voda Cantacuzino (1679 – 1688), has done at Calugareni a lasting bridge to commemorate the Calugareni victory of Mihai The Brave against the Turks; I quote: „ ... a nice and wonderful bridge ... great and thoroughgoing bridge to eternal remembrance"

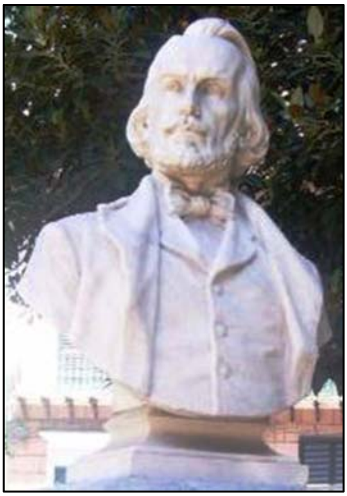

Figure 8. Statue of Nicolae Balcesu

Until towards the end of XVIIth century, the term bridge builder was given three meanings by Bucharesters:

1. Those people involved in bringing about the continuity of a road over an obstacle, usually a water course
2. Those people involved in building pavements made of wood, or of oak tree trunks, which would cover a main road
3. People that suffered from plague but survived, and were asked to search the attics of homes to discover people still suffering from plague (N.B.: the Romanian word for „attic" is „pod"; the Romanian word for „bridge" is „pod" too, the two notions being homonymous)

These bridge people (1) should not be confused with those who would build bridges on dry land (2)! The latter would not do anything but placing oak tree trunks, roughly wrought by chopping, along main roads, on top of which wooden planks were placed transversally which would have the function of a floor on which wagons and carriages would travel. Fig 3-15

The justification of this solution was given by the idea of protecting the vehicles from dust and mud. It was a false idea. In the cross section of the road, along the centerline, under the wooden pavement, there was a ditch destined for collecting and draining the water, in which debris and mud would accumulate. It is not difficult to imagine what kinds of miasmas were yielded in time by their existence along the road. Nicolae Iorga recorded the existence of these „bridges on land" yet from 1574.

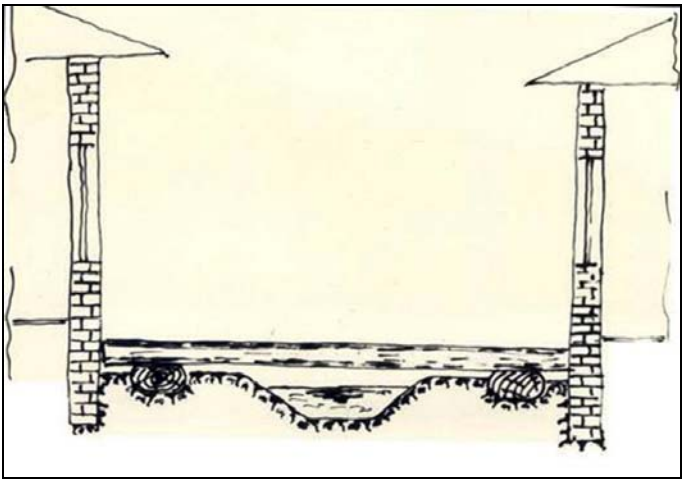

Figure 9. Cross section of a road used by aristocracy in Bucharest

The existence of such bridges on dry land was probably possible due to the fact the area was covered by Vlasiei forest.

Hence, the name given to streets, not only of Bucharest, i.e. bridges. History records the same term for the city of Iasi. We enumerate, hereinafter, the principal such „bridges":

1. The aristocracy bridge of The Main Street. In the year 1574 the Frenchman Pierre Lescalopier, passing by Bucharest, in June, mentioned that „pavement is made of tree trunks".
2. The Great Bridge, Serban Voda Bridge and Beilicului Bridge, al are sequential names given to today's Calea Serban Voda;
3. Calicilor (Beggers') Bridge, today's Calea Rahovei;
4. Targul de Afara (or Outdoor Market) Bridge, today's Calea Mosilor;
5. Mogosoaia Bridge, today's Calea Victoriei, and others;
6. Beilicului Bridge, today's Calea Serban Voda.

Figure 10. Mihai Voda Architectural Complex, Mihai Voda Bridge, 1794.
History Museum of the City of Bucharest. Reproduction from a chromo-lithography of Luigi Mayer's water color painting

In Iasi we find names like the Long Bridge, the Old Bridge, the Green Bridge.

In 1807, the Englishman Thornton noticed that, I quote: „the waste of such a fine wood material, which should be replaced every 5 to 6 years, cannot be justified by any need".

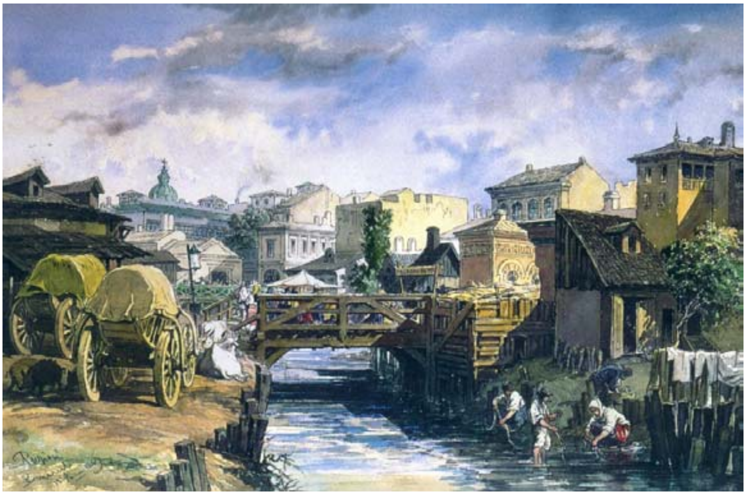

Figure 11. Flower market and the bridge over Dambovita river; XVIth – XVIII centuries

For city streets the term stone pavement, subsequently become cobble stone pavement, has appeared only in the XVIIIth century, and replaced the so called bridges on dry land.

There are no data available about the wooden bridges which existed over Dambovita river, confirmed by historic documents. The two chromolithographic images preserved by the Museum of the City of Bucharest, give us some information about the evolution of this type of bridges in this area.

References

1. Iorga, N., Istoria lui Ştefan cel Mare, Ed. Minerva, Bucureşti, 1978;
2. Giurescu, Dinu C., Istoria ilustrată a românilor, Ed. Sport – Turism, 1981;
3. „Letopisețul de la Putna”. Cronică în limba slavonă. Prezintă evenimentele petrecute în istoria Moldovei între 1359 şi 1518;
4. Simanschi, Leon ş.a., Documenta Romania Historica A Moldova, vol. II (1449-1486, Ed. Acad. RSR, 1976;
5. Ionescu, I., Podurile, Buletinul Societății Politehnice, vol. I, Bucureşti, 1931;
6. Istrate, C. I., Biserica şi podul din Borzesci precum şi o ochire relativă la bisericile zidite de Ştefan cel Mare, Analele Academiei Române, Seria II – Tomul XXVI, 1903-1904.

Theoretical aspects for arch bridge calculation

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Summary

This paper presents a theoretical method for arch bridge calculation using the static of construction.

In determining dead loads sectional efforts arc calculation was done by transferring the unknowns in central elastic.

The effect of moving loads is determined using influence lines. To simplify the calculation it was used the direct transmission influence lines hypothesis for mobile loads. Calculation of effort is made in specific sections of the arc (birth, quarter key).

KEYWORDS: arch, elastic center, influence lines.

1. INTRODUCTION

Currently, in Romania, are operating several road and railway bridges and aqueducts on arches and vaults.

Bridges on arcs occupy an important place in the history of bridges. Over time bridges were built that were outstanding as main structure resistant vault, from the vault of stone bridges made by the Romans to modern bridges to arches made today. Depending on the material it is made of resistant structure bridges the arches, and the location, the nature of the terrain, the possibilities of mounting openings that can be covered by this category of structures reach 300m values for the arches of bridges concrete, ie 550-600m for bridges on metal arches.

Reinforced concrete begins to make its presence in the construction of bridges in Romania after years 1905-1906, placing the concerns of Romanian engineers at the forefront of global concern. Literature states that the first bridge was concrete bridge in Sinaia in vaults solution with tambourines made in 1899.

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2. TEORETICAL ASPECTS FOR ARCH CALCULATION

In this study, statically indeterminate arcs must take into account that the axis is curved bar. Statically indeterminate structures axle and thrust curve used in construction are double arch recessed arch with two hinges, bow tie, continuous arcs.

Section height distribution is normal unit effort by a law of variation hyperbolic.

2.1. Classification of arcs used as resistance structure in construction

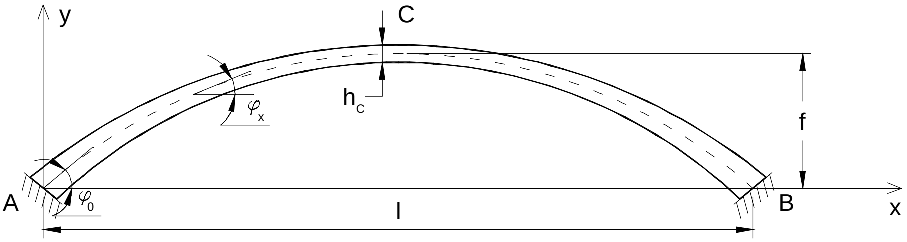

Figure 1 – Double restrained arch

Depending on the arrow f, and l arch opening, arcs are classified:

1. drooping arches - which $f \leq \frac{l}{5}$;
2. large arrow arches - which $f > \frac{l}{5}$

Depending on the key section height hc and opening it arch, arcs are classified:

3. thick arcs - which $\frac{l}{10} > h_c > \frac{l}{30}$
4. thin arcs - which $h_c \leq \frac{l}{30}$

2.2. Solving double arch embedded

Calculation of reinforced concrete bridge deck in the bow, which connects arc deck and is achieved by uprights, is usually in the following simplifying assumptions:

- To calculate arc supposedly deck beam is hinged to each post so that the system above is considered static arch determined statically indeterminate arch not cause the calculation. In this case the role of deck beams and pillars is reduced only to send stress to arch

- For the calculation of deck beam is considered very stiff spring and therefore neglecting deformations spring deck beam is treated as a continuous beam on fixed supports right-pillars.

Embedded double arches are usually done with variable section, considering the following law of variation of the moment of inertia: $I_x = \dfrac{I_c}{\cos\varphi_x}$

2.2.1. Static loads

Understand the importance of the law of variation of the cross section of the arc along the opening behavior is conditioned by the knowledge and calculation of these types of resistance elements.

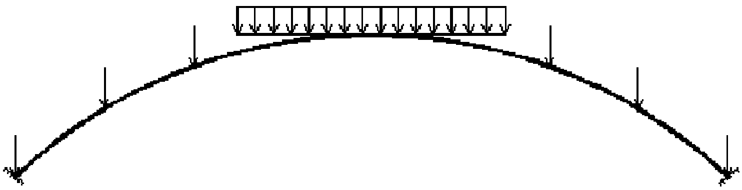

Figure 2 – Loading scheme

Embedded double arches are statically undetermined structures triple. The base is obtained by deleting three links and introducing three unknowns Xi efforts. The three unknown undetermined X1, X2, X3 are obtained from expressing any form of basic continuity.

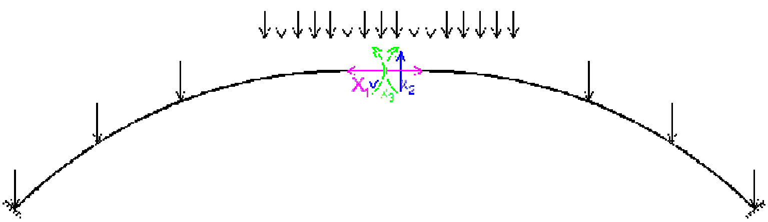

Figure 3 – Base system

Because our system is a closed contour, recessed unknowns separated when placed in the center of the arch elastic, using arms considered infinitely rigid.

$$\begin{cases} \delta_{11}\cdot X_1+\delta_{12}\cdot X_2+\delta_{13}\cdot X_3+\Delta_{1p}=0 \\ \delta_{21}\cdot X_1+\delta_{22}\cdot X_2+\delta_{23}\cdot X_3+\Delta_{2p}=0 \\ \delta_{31}\cdot X_1+\delta_{32}\cdot X_2+\delta_{33}\cdot X_3+\Delta_{3p}=0 \end{cases}$$

$\delta_{ij}-$ Unknown X direction displacement produced by a reaction equal to unity on unknown X

$\Delta_{ip}-$ movement towards the unknown X, produced by external loads

Unknown's coefficients and free terms of the equations of equilibrium are determined by Mohr-Maxwell relations.

$$\delta_{ij}=\sum\int\frac{N_i\cdot N_j}{EA}ds+\sum\int\frac{M_i\cdot M_j}{EI}ds+\sum k\int\frac{T_i\cdot T_j}{GA}ds+\sum\int M_i\cdot\frac{1}{\rho}\cdot\frac{N_j}{EA}ds+\sum\int N_i\cdot\frac{1}{\rho}\cdot\frac{M_j}{EA}ds$$

If the spring has an axis of symmetry, basic shape also choose a symmetric, where it was severed arm hypothetical rigid entered.

For reasons of symmetry, separated by unknown X1, X2 and X3. X1 to X3 is separate severing the arm stiff enough to make the heart leading to elastic $\delta_{13}=\delta_{31}=0$.

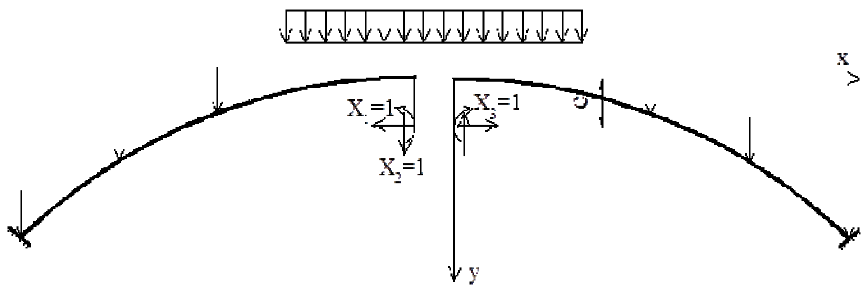

Figure 4 – Basic system with elastic center

If we consider xOy axis system with origin at a point one axis of symmetry resulting:

$$M_1=-y,\ N_1=-\cos\varphi,\ M_2=x,\ N_2=-\sin\varphi,\ M_3=1,\ N_3=0$$

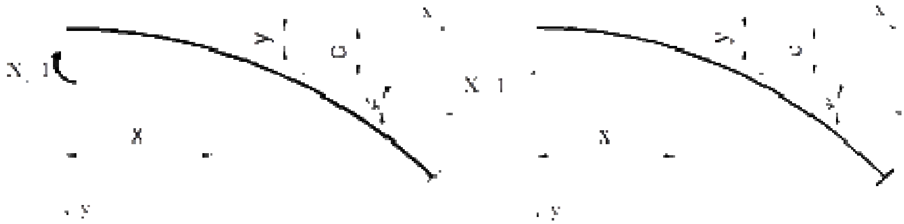

Figure 5 – Sectional efforts for a half arch

Condition $EI_0\delta_{13} = \int \frac{I_0}{I} m_1 m_3 ds + \int \frac{I_0}{A} n_1 n_3 ds = -\int \frac{I_0}{I} yds + 0 = \int ydw = 0$ defines a point O on the axis of symmetry as the center of gravity of elastic loads $dw = \frac{I_0}{I} ds$, called elastic center. Its equation is obtained from the static moments about the horizontal axis elastic loads Cx1-key.

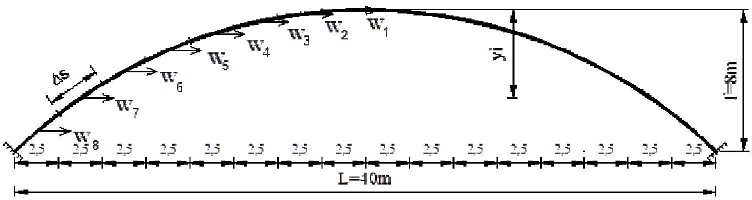

Figure 6 – Coordinates for elastic center

Results $c = \frac{\int y_1 dw}{\int dw}$, where y_1 is the current vertical ordinate axis points arch about the horizontal axis C_{X1} and c is the distance from the center axis elastic 0. Integrals can be half the arc.

For numerical calculation, when the integrals are difficult, is divided into bricks arc length Δs, Elastic concentrated loads are introduced $W = \frac{I_0}{I} \Delta s$ and the expression becomes $c \approx \frac{\sum W \cdot y_1}{\sum W}$

Later computations are carried out with axes originating in an elastic center leading to equations of condition:

$$\begin{cases} \delta_{11} \cdot X_1 + \Delta_{1p} = 0 \\ \delta_{22} \cdot X_2 + \Delta_{2p} = 0 \\ \delta_{33} \cdot X_3 + \Delta_{3p} = 0 \end{cases}$$

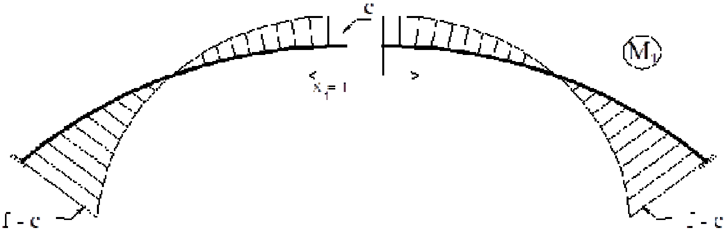

Figure 7 – Moment diagram from X_1

$$\delta_{11} = \int_0^l \frac{M_1 \cdot \overline{M_1}}{EI} dx + \int_0^l \frac{N_1 \cdot \overline{N_1}}{EA} dx = \frac{1}{EI_0}\left(\int_0^l \frac{I_0}{I} y^{/2} dx + \int_0^l \frac{I_0}{A} \cos^2 \varphi dx \right) = \frac{1}{EI_0} \int_0^l \frac{I_0}{I} y^{/2} dx + i_0^2 l$$

If the spring is divided into bricks relationship becomes

$$\delta_{11} = \frac{1}{EI_0}\left(\sum_{i=1}^{n} W_i \cdot y_i^{/2} + i_0^2 l \right)$$

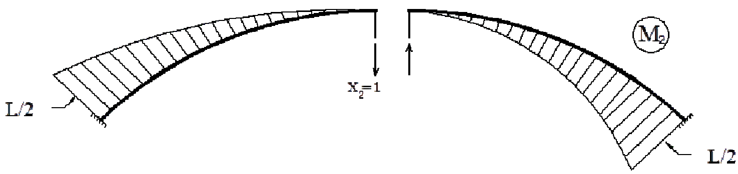

Figure 8 – Moment diagram from X_2

$$\delta_{22} = \int_0^l \frac{M_2 \cdot \overline{M_2}}{EI} dx + \int_0^l \frac{N_2 \cdot \overline{N_2}}{EA} dx = \frac{1}{EI_0}\left(\int_0^l \frac{I_0}{I} x^2 dx + \int_0^l \frac{I_0}{A} \sin^2 \varphi dx \right) = \frac{1}{EI_0} \int_0^l \frac{I_0}{I} x^2 dx$$

If lop arc angle φ is small that sinφ →0 so the second term is canceled.If the spring is divided into bricks relationship becomes

$$\delta_{22} = \frac{1}{EI_0} \sum_{i=1}^{n} W_i x_i^2$$

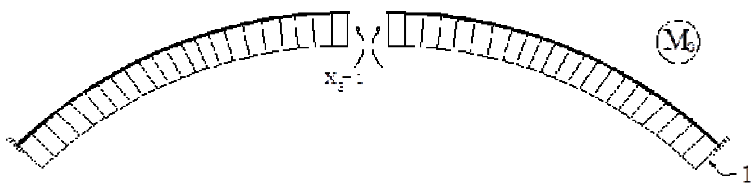

Figure 9 – Moment diagram from X_3

$$\delta_{33}=\int_0^l \frac{M_3\cdot\overline{M_3}}{EI}dx+\int_0^l \frac{N_3\cdot\overline{N_3}}{EA}dx=\frac{1}{EI_0}\left(\int_0^l \frac{I_0}{I}1^2dx+0\right)$$

$$\delta_{33}=\frac{1}{EI_0}\int_0^l \frac{I_0}{I}1^2dx$$

If the spring is divided into bricks relationship becomes

$$\delta_{33}=\frac{1}{EI_0}\sum_{i=1}^{n}W_i$$

Form the basis for external loads to choose independent basic forms of unknowns. In general, the basic form caused some static, external forces will cause bending moments M_0 and axial forces N_0. Free terms Δ1p, Δ2p, Δ3p the continuity equations are expressions

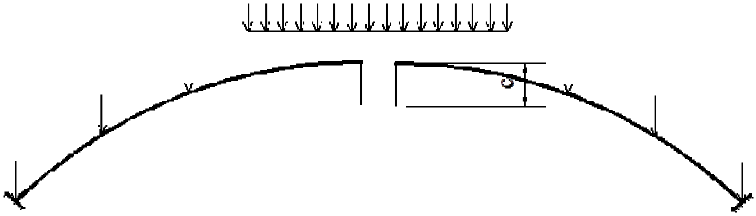

Figure 10 – Base system under by permanent loads

$$\Delta_{1p}=\int_0^l \frac{M^0\cdot\overline{M_1}}{EI}dx+\int_0^l \frac{N^0\cdot\overline{N_1}}{EA}dx=\frac{1}{EI_0}\left(\int_0^l \frac{I_0}{I}M^0\left(-y'\right)dx+\int_0^l \frac{I_0}{A}N^0\left(-\cos\varphi\right)dx\right)$$

$$\Delta_{1p}=\frac{1}{EI_0}\int_0^l \frac{I_0}{I}M^0\left(-y'\right)dx=-\frac{1}{EI_0}\sum_{i=1}^{n}W_i\cdot M_i^0\cdot y_i'$$

Considering that spring is flattened second term is very small and may be neglected.

$$\Delta_{2p} = \int_0^l \frac{M^0 \cdot \overline{M_2}}{EI} dx + \int_0^l \frac{N^0 \cdot \overline{N_2}}{EA} dx = \frac{1}{EI_0}\left(\int_0^l \frac{I_0}{I} M^0 x dx + \int_0^l \frac{I_0}{A} N^0 \sin\varphi dx \right)$$

$$\Delta_{2p} = \frac{1}{EI_0} \int_0^l \frac{I_0}{I} M^0 x dx = \frac{1}{EI_0} \sum_{i=1}^{n} W_i \cdot M_i^0 \cdot x_i$$

x_i – is the distance from the middle of bricks i and the ordinate axis y.

$$\Delta_{3p} = \int_0^l \frac{M^0 \cdot \overline{M_3}}{EI} dx + \int_0^l \frac{N^0 \cdot \overline{N_3}}{EA} dx = \frac{1}{EI_0}\left(\int_0^l \frac{I_0}{I} M^0 x dx + 0 \right)$$

$$\Delta_{3p} = \frac{1}{EI_0} \int_0^l \frac{I_0}{I} M^0 1 dx = \frac{1}{EI_0} \sum_{i=1}^{n} W_i \cdot M_i^0$$

M_i^0 – value of load external bending moment and mid bricks i.

$$\begin{cases} X_1 = -\dfrac{\Delta_{1p}}{\delta_{11}} \\ X_2 = -\dfrac{\Delta_{2p}}{\delta_{22}} \\ X_3 = -\dfrac{\Delta_{3p}}{\delta_{33}} \end{cases}$$

After determining the unknown's efforts arch sections are calculated as a console with curved axis.

$$M = M_p + X_1 \cdot M_1 = M_p - X_1 \cdot y$$
$$N = N_p + X_1 \cdot N_1 = N_p - X_1 \cdot \cos\varphi$$

2.2.2. Arch under mobile loads

The effect of moving charges are determined using influence lines. since the unknowns X1, X2, X3 the elastic center is determined from independent equations, the most common method for determining lines of influence, is to first determine influence lines of unknowns X1, X2, X3 which then deduced by overlapping effects.

$$\begin{cases} X_1 = \dfrac{\delta_{i1}^*}{\delta_{11}} \\ X_2 = \dfrac{\delta_{i2}^*}{\delta_{22}} \\ X_3 = \dfrac{\delta_{i3}^*}{\delta_{33}} \end{cases}$$

δi1* – is the elastic displacement diagram form the basic mobile workforce P=1 in load X1 = 1

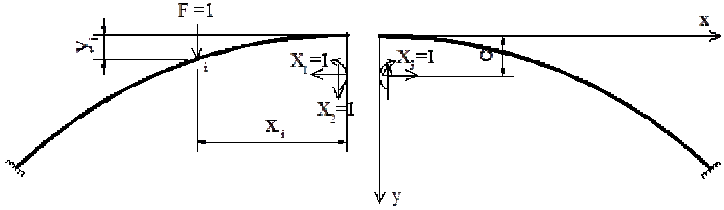

Figure 11 – Base system. Mobile loads

Determine the elastic lines (diagrams movements) δi1* is determined by elastic loads and combined systems.

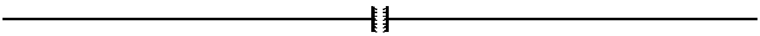

Figure 12 – Base system – conjugated

If we consider the basic shape console system consists of two key obtained by cutting the arch symmetrically coupled system is two consoles Embedded in sections of the axis of symmetry. For this basic form (two consoles) lines of influence of all unknowns X1, X2, X3, have ordered null and void slopes in sections births. The virtual displacement, suppression by one of the links directions unknowns X1, X2, X3, embedding does not change, which does not allow any movement or rotation for deformations.

Influence lines for mobile loads

For X_1 the relation is: $X_1 = \dfrac{\delta_{i1}^*}{\delta_{11}} = \eta_{1i}$

$\delta_{i1}^{*} = M_i^f -$ is the diagram of moments burdens elastic joint beam produced by charge X_1

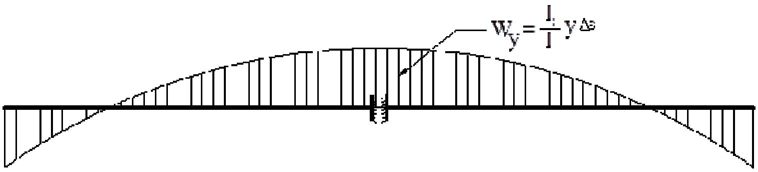

Figure 13 – Conjugated system under elastic loads for X1

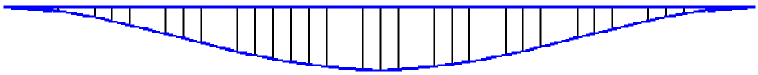

Figure 14 – Influence line for X1

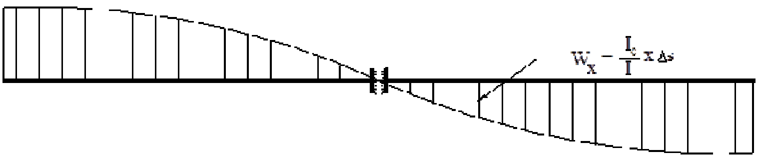

Figure 15 – Conjugated system under elastic loads for X2

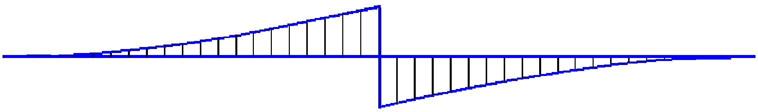

Figure 16 – Influence line for X2

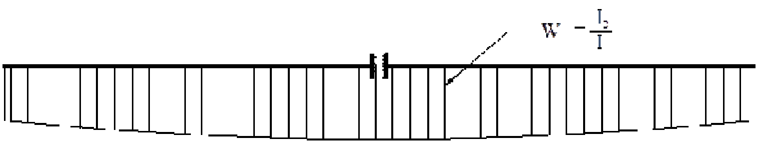

Figure 17 – Conjugated system under elastic loads for X3

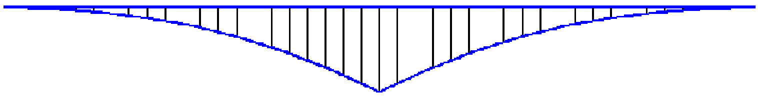

Figure 18 – Influence line for X3

Influence of efforts sectional lines are drawn by overlapping effects on the expression of these efforts statically undetermined depending on the size and the action P = 1 mobile.

In general, an effort Si:

$$\eta_{S_i} = \eta_i^0 + \eta_1 S_i^1 + \eta_2 S_i^2 + \eta_3 S_i^3$$

η_i^0 – ordinate of influence line based system (statically determined),

η_1, η_2, η_3 – ordinates of influence lines of statically undetermined quantities,

$S_i^{(1)}$, $S_i^{(2)}$, $S_i^{(3)}$ – effort values S_i products based system actions $X_1 = 1;\ X_2 = 1;\ X_3 = 1$

Determine lines of influence within moments the central core M_{ki}, M_{ks} for sections, key quarter and birth.

For vertical section is marked with y_{ki} and y_{ks} distances from the axis passing through the center of the central core elastic limits

$$M_{ki} = M^0 - X_1 y_{ki} + X_2 x_{ki} + X_3$$
$$M_{ks} = M^0 - X_1 y_{ks} + X_2 x_{ks} + X_3$$

Knowledge of moment to limit central core effort determines the extreme fiber of the section.

Influence lines, the bending moment in section birth, fourth and key

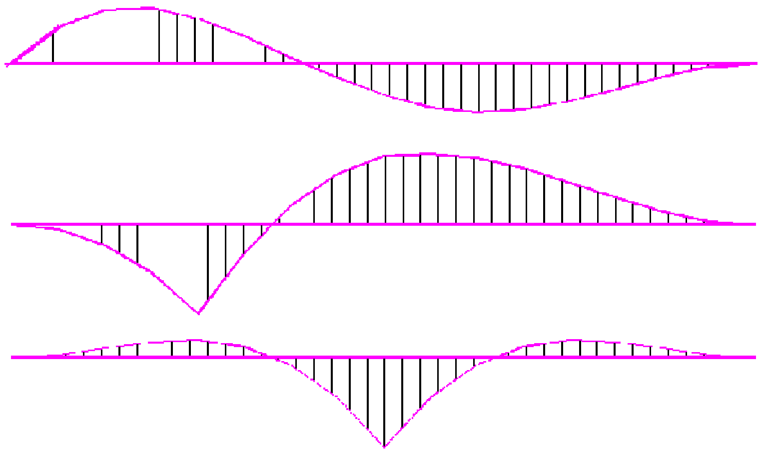

Figure 18 – Influence line mobile loads

To determine the maximum load moments moving parts convoy has load line so that the value is multiplied by concentrated forces ordered on line bigger influence.

3. CONCLUSIONS

Calculation of arches some specific aspects perform. Adoption of the arches resistance structures brings clear advantages in exploiting of bridges structures. Operating benefits are reduced calculation complexity of the design stage, computational engineers often raises problems.

Analysis of the viability of the arches of bridges in operation on road and railway network, good behavior is observed over time, and rehabilitation solutions are available, ensuring the transition to Class E cargo, the only drawback remains in this stage, checking bearing capacity.

References

1. Clough, R.W., Penzien, J., *Dynamics of structures*, McGraw-Hill, New York, 1993.
2. Achkire, Y., Preumont, A., Active tendon control of cable-stayed bridges, *Earthquake Engineering and Structural Dynamics*, vol. 25, 1996.
3. Popescu, I., Tanase, R., Asupra modelării statistice a fenomenului de defectare în cazul podurilor, *Buletinul IPI*, Tomul XLVII, Fasc. 5, 2001. (in Romanian)
4. Magaña, M.E. and Rodellar, J., Nonlinear decentralized active tendon of cable-stayed bridges, *Journal of Structural Control*, vol. 5, 1998.
5. Păunescu, F., Badea-Dincă, N., Stăcuţ., E. *Informatizarea societăţii: un fenomen inevitabil ?*, Ed. Şiinţifică şi Enciclopedică, Bucureşti, 1985. (in Romanian)

Preliminary study for conception, assimilation and implementation of long lasting flexible pavements in Romania

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Summary

The actual flexible pavements designed according the existing norms are leading usually to overdesigned structures because of the lower values for the elastic modulus of the asphalt materials, specified in the existing norms. After a short introduction, presenting the general principles of flexible pavements design, the concepts of long lasting flexible pavements is considered in detail. Then a new research program, involving Accelerating Loading Test-ALT undertaken in parallel with the experiments development on the existing road network, is proposed to the attention of the road policy decisions factors in this country. This research project, supported by specific design assumptions and calculations is taking into considerations the specific soil, climatic and traffic conditions of the road network in Romania. Finally a discussion of the results obtained with this new study is made.

KEYWORDS: Long lasting flexible pavements, accelerating loading test - ALT, structural design, design methods, design traffic.

1. INTRODUCTION

As a member of European Community, Romania is making significant efforts to integrate his transport infrastructure in the huge European road network by using new modern and efficient methods for structural design in parallel with the implementation of new construction technologies. Our research is dedicated mainly to the assimilation and the development, in the specific traffic and climatic conditions of Romania characterized by very severe winters and very hot summers, of the new concept of long lasting pavements, especially for the constructions of the new roads and motorways. The actual flexible pavements designed according the existing norms [1] are usually leading to overdesigned structures because of the lower values for the elastic module of the asphalt materials specified in the existing norms. The total thickness of classical pavement structures, currently used for important motorway projects in this country, is currently reaching significant values ranging from 75 to 95 cm. In comparison with these traditional practices the long lasting flexible pavements LLFP, conceived on new principles and involving the use of high quality materials such as stone matrix asphalt SMA [2] are leading to thinner and in the same time more durable pavements. Here follows some typical example of LLFP structures envisaged to be studied on the Accelerating Testing Facility ALT-LIRA [3] existing in the frame of Technical University Gh. Asachi Iasi (see Fig.1).

ACCELERATED LOAD TRACK - ALT-LIRA

Manufacutred year:	1982
Measure:	
Arms lenght:	15
No. of arms:	2
Load:	
Axle:	115 kN
Transverse distribution:	2
Axle configuration:	

Figure1. The ALT circular track facility of Technical University “Gh. Asachi” Iasi [3]

In order to evaluate the performance of these long lasting pavements compared to the conventional one, a new experiment involving accelerated load testing of eight distinct pavements sectors, four of which are conventional (No.1, No.2, No.3 &No.4) and other four being design as long lasting pavements (No.5, No.6, No.7 & No.8). In designing these sectors, different values for design traffic varying between 5 & 60 msa (million standard axle) for design life of 15 30 & 45 years, have been envisaged.

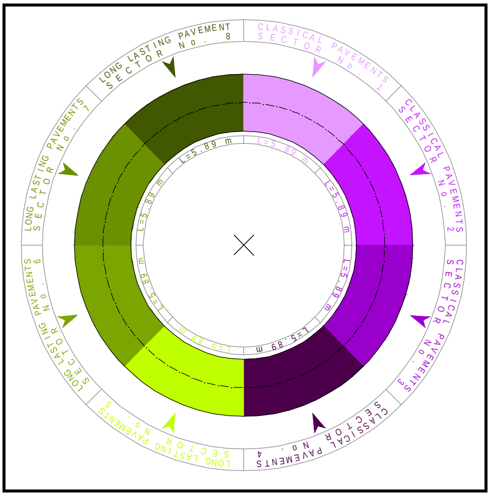

Figure 2. Experimental sectors envisaged to be tested on the ALT circular track facility

For long lasting pavements to be viable, they must perform from the perspectives of both engineering and economic consideration. Designing against structural defects, proper materials selection, good construction practices, and scheduling resurfacing activities to maintain the functionality of the pavement are the primary engineering concerns for their performance [4]. Efficient design, low maintenance rehabilitation costs, and long pavement life will ensure the economy of the pavement. In accordance with long lasting pavement concept, it is necessary to periodically monitor the pavement condition in order to identify surface distresses and to ensure they do not further progress into the structure than the top few cm of the pavement. Thus, distresses such as top-down fatigue cracking, thermal cracking, rutting, and surface wear can be confined only to the wearing course by timely resurfacing. According existing literature there are a number of case histories [4] that support the idea that thick, well-constructed asphalt pavements have distresses extending no deeper than their surfaces. The future work involves the construction of the envisaged experimental sectors on the circular track of the ALT facility of Technical University of Iasi, parallel with the construction of similar experimental sectors on a real motorway selected on the existing public road network, followed by monitoring their performances in time and the drafting of specific technical recommendations for the design and constructions of LLFP.

2. STRUCTURAL DESIGN OF FLEXIBLE PAVEMENTS

In this study two types of pavement structure have been considered, a classical one and a Long Lasting Flexible Pavement conceived according the principals mentioned above. This design study has been conducted according the Romanian norm PD 177/2001 which is based on simultaneously observance of the following criteria:

- the admissible tensile strain at the bottom of the bituminous layers;
- the admissible compression strain at the subgrade level;
- the admissible tensile strain at the bottom of the layer of natural aggregates stabilized with hydraulic or pozzolan binders.

This analytical design method involves the establishing of a specific road pavement structure and verification of the loading conditions of pavement, under the design traffic and also frost verifications.

The following input data are necessary for the design:

- structure and intensity of traffic and their evolution;
- the geotechnical characteristics of the subgrade;
- the hydrological regime of the road pavement (type of cross section, the way of rainfall waters drainage, possibilities of drainage, level of ground water).

2.1. Conception and design of classical/witness ALT sectors

The experimental road sector envisaged for study was considered to be located in a climatic region type I, having cross sections in embankment with a maximum height of 1.00 m, the subgrade soil being a P5 type according [1]. In this hypothesis, and considering three categories of design traffic expressed in million standard axles (msa), namely: 5, 10, 30 and respectively 60 msa, the following pavement structures have been studied:

Tabel 1. The classical pavement structure proposed for the design

CLASSICAL PAVEMENTS						
Layer type	No. 1	No. 2	No. 3	No. 4	E [MPa]	μ
	h [cm]	h [cm]	h [cm]	h [cm]		
Wearing (MASF 16m)	5	5	5	5	4000	0.35
Binder (BAD 25m)	8	8	10	10	3500	0.35
Bituminous base (AB2)	10	10	15	15	5000	0.35
Ballast stabilized with cement	15	20	20	30	1000	0.25
Ballast foundation	30	30	35	35	208/208/223/223	0.27
Subgrade/Soil type	P5	P5	P5	P5	80	0.42
Total thickness	68	73	85	95	-	-

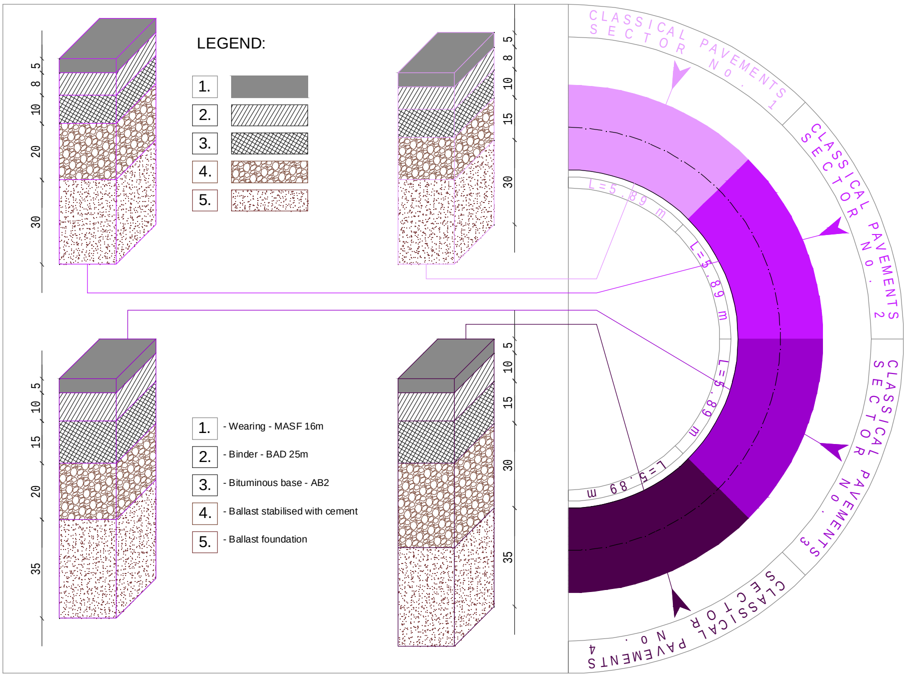

Figure 3. Proposed experimental classical sectors on the ALT circular track facility

2.2. Conception and design of Long Lasting Flexible Pavement ALT sectors

The road sector envisaged for the study of LLFP has been considered but the design was conducted for same categories of traffic, namely: 5, 10, 30 and respectively 60 m.s.a. The following LLFP pavement sectors have been conceived and verified according the PD 177-2001 procedures:

Tabel 2. The long lasting flexibile pavement structure proposed for the design

LONG LASTING FLEXIBLE PAVEMENTS						
Layer type	No. 5 h [cm]	No. 6 h [cm]	No. 7 h [cm]	No. 8 h [cm]	E [MPa]	μ
Upper – SMA/MASF	5	5	5	5	7000	0.35
Intermediate – AVS*	20	25	30	30	6000	0.35
Bottom – SMA	5	5	5	5	7000	0.35
Ballast Foundation	25	25	30	35	192/192/208/250	0.27
Subgrade/Type soil	P5	P5	P5	P5	80	0.42
Total thickness	55	60	70	75	-	-

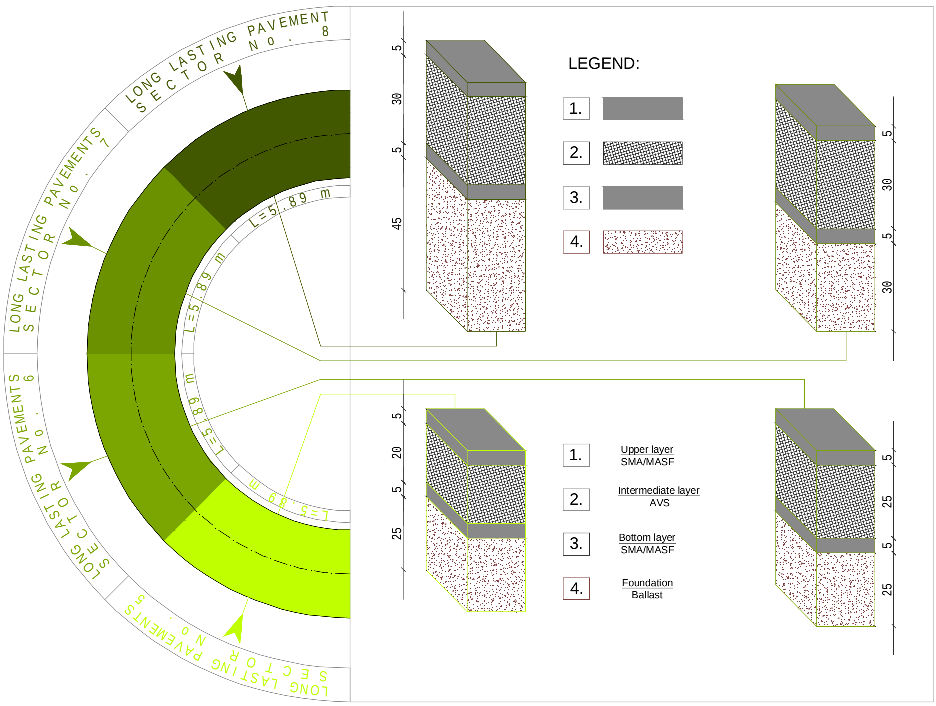

Figure 4. Proposed experimental long lasting sectors on the ALT circular track facility

3. CHECKING THE FROST RESISTANCE OF THE PAVEMENTS

The pavement structure was design as way to resist at freezing and thawing conditions the positions of the existing Romanian Standards [5], [6], [7]. The results of their verification are given in Table 3.

- Subgrade soil type: P5
- Climatic type: I
- Hydrological regime: 1
- Level of underground water Naf : 300 cm
- Depth of freezing: 70 cm

Table 3. Frost verification of flexible pavement structure proposed for the design

Crt. Nr.	Thaw-Freze Check	Number of pavement structures on track facility test							
		No. 1	Nr. 2	Nr. 3	Nr. 4	Nr. 5	Nr. 6	Nr. 7	Nr. 8
1	$H_{S.R.}$ [cm]	68	73	85	95	55	60	70	85
2	$H_{ech.}$ [cm]	43.05	46.3	53.5	60	40.5	43.5	50.5	59.5
3	ΔZ [cm]	24.95	26.7	31.5	35	14.5	16.5	19.5	25.5
4	Z [cm]	65	65	65	65	65	65	65	65
5	$Z_{cr.}$ [cm]	89.95	91.7	96.5	100	79.5	81.5	84.5	90.5
6	$K_{ef.}$	0.48	0.50	0.55	0.60	0.51	0.53	0.60	0.66
7	$K_{adm.}$	0.4	0.4	0.4	0.40	0.50	0.50	0.5	0.5
$K_{ef.} \geq K_{adm.}$		**0.48 > 0.40**	**0.50 > 0.40**	**0.55 > 0.40**	**0.60 > 0.40**	**0.51 > 0.40**	**0.53 > 0.50**	**0.60 > 0.50**	**0.66 > 0.50**

4. CONCLUSIONS

As a final synthetic comparative analysis of performances obtained at this research stage for classical and long lasting flexible pavements design with PD 177/2001 for roads with medium and heavy traffic we may conclude the following:

- For pavements structures design for 15 years, all the design criteria verified, according PD177/2001 recommendation, are fulfilled;
- For pavements structures designed for 30 years the design criteria has been partially fulfilled as are listed in Table 4:

Table 4. Analysis of the design criteria fulfilled for a pavements structures life time of 30 years

Design Criteria	CLASSICAL				LLFP			
	No.1	No.2	No.3	No.4	No.5	No.6	No.7	No.8
tensile strain at the bottom of the bituminous layers	NO	NO	NO	NO	OK	OK	OK	OK
tensile strain at the bottom of stabilized aggregates	OK	OK	OK	OK	NN	NN	NN	NN
compression strain at the subgrade level	NO	NO	NO	NO	OK	OK	OK	OK

*Note: OK - pavement structures meets the design criteria; NO - pavement structures does not meet the design criteria; NN - not necessary verification;

- For pavements structures designed for 45 years the design criteria has been partially fulfilled as are listed in Table 5:

Table 5. Analysis of the design criteria fulfilled for a pavements structures life time of 30 years

Design Criteria	CLASSICAL				LLFP			
	No.1	No.2	No.3	No.4	No.5	No.6	No.7	No.8
tensile strain at the bottom of the bituminous layers	NO	NO	NO	NO	NO	OK	OK	NO
tensile strain at the bottom of stabilized aggregates	OK	OK	OK	OK	NN	NN	NN	NN
compression strain at the subgrade level	NO	NO	NO	NO	NO	NO	NO	NO

We may conclude that in comparison with the conventional pavements, the long lasting ones, conceived and designed according principals of LLFP have proved to meet all the design criteria and are capable to support the design traffic twice bigger corresponding for a design life of 30 years.

Further research is envisaged to be undertaken into the future, involving the verifications of designs of both types of pavements by using the Mechanistic-Empirical Pavement Design Guide [14][15].

References

1. Romanian Doc. PD 177-2001. Recommendation for the design of flexible and composite pavements, 2001
2. Andrei R.& others, Technical recommendation for the asphalt mixes stabilized with cellulose fibers , AND 539-2002.
3. Andrei R., Options on LLRP experimental sectors intended to be constructed and tested on the ALT circular track facility of Technical University "Gh. Asachi" Iasi, www.ecolanes.com, STREP FP6 project, 2009.
4. Asphalt Pavement Alliance, www.asphaltalliance.com,
5. Vlad N., STAS 1709/1-90, Frost depth in pavement road. 1990.
6. Vlad N., STAS 1709/2-90, Prevention and reparation of frost-thaw damages 1990.
7. Vlad N., STAS 1709/3-90, Determination of sensitivity to frost of the soil – Method of test, 1990.
8. National Highway Institute, "Introduction to Mechanistic-Empirical Design of New and Rehabilitated Pavements", 2002.
9. R. Andrei & others: Climatic and ecological aspects of structural design of Long Lasting Rigid Pavements – LLRP for demonstration projects located in different European regions, ISSN: 1790-2769, ISBN: 978-960-474-182-3, 2010.
10. Montalvo, J., & others : A New Ecological Pavement For Enhancement Of Urban Environments, **WSEAS** international conference on energy, environment, ecosystems and sustainable development, July 12-14, 2005
11. Rajab M & others, Application Of Image Processing To measure road distresses WSEAS transactions on information science & applications volume 5, 2008
12. Kaur D., Soft Computing Technique In Prediction Of Pavement Condition, 6th WSEAS int. conference on computational intelligence, man-machine systems and cybernetics, Tenerife, Spain, December 14-16, 2007
13. Shandiz H. T., & others , Using Multi Layer Perceptron Network to Classify Road Cracks, Proceedings of the 6th **WSEAS** int. conf. on neural networks
14. http://onlinepubs.trb.org/onlinepubs/archive/mepdg/home.htm
15. Tanasele I., Teza de doctorat. Studiul privind conceptia si executia unor structuri rutiere flexibile durabile. 2012.

Dams build up from local material

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Summary

In the last decades, the environmental problem, together with the bad exploitation of the dams with the main purpose of fishing or defending against flooding has worsened the dams state so bad, that the safety exploitation and structural integrity are no longer insurance without a proper rehabilitation.

Nowadays, the usage of renewable energy context, gives an important opportunity to rehabilitate the dams with a complex value of water resources. The places where high dams could be built are exhausted, so the usage of the hydraulic energy from dams of short or medium height take's an important role.

A proper construction or rehabilitation of a dam should always be a function of the level of infiltration through dam body, the internal forces state and the general stability at live loads and earthquake effect.

Concerning the infiltration level, many methods of rehabilitation have been developed through the last years, methods of infusion of different materials or methods which involve usage of different bearing elements.

It is a present tendency to use geotextiles in the rehabilitation of soil dams, because of the short amount of work and cheep solution of raise.

The treatment of foundation soil, together with the usage of composite materials, leads to a safety construction and an economical one.

The following work aims to combine the modern with the previous year's practice to capitalize the power and protection of dams from local materials through rehabilitation, improvement of soil foundation, waterproofing, etc.

KEYWORDS: dams, foundations, permeable soils, mellow sands, drifts, seepage, intrusion, methods.

1. INTRODUCTION

From early beginnings, dams have been considered a fascinating area. At first, these constructions were created using the most accessible material, soil. Examples can be found in Jordan, Egypt, Jawa or Sadd-el-Kafara, [1] dating over 4800 years ago. Later archeological discoveries identify dams raised up over 3000 years ago on rivers Amu-Daria and Sir-Daria; roman dams built at the beginning of our era; Persian, Indian or Japanese dams around 1000 years before Christ.

By the end of 19th century there is recorded a new stage in ground dams execution due to perfecting the compaction technique by using smooth cylinders and compactor cylinders. Thanks to mechanization development, during the last 50 years, there were realized improved and ingenious ground dams with advantageous price execution.

In Romania, the oldest proof of this type of constructions belongs to the Transylvanian area.

The dams build with local materials eliminates the problem of material transportation, usage of expensive technologies and the deficient difficult materials. These are, in most cases, optimum solutions from technological and economical point of view.

2. FOUNDATION SOILS

The basis for any construction work is represented by the foundation. The infrastracture design implies accomplishing the safety and resistance criteria. The experts in civil engineering must take into consideration the way soil behaves as material, support structure and construction component.

In different countries, the engineering experience has made possible, from ancient time, the usage of local material (rocks and soils) for infrastructure construction works. For the particular case of Romania might be named the Transfăgărăsan Road.

In time, due to technological progress, the usage of local materials has been extended, currently being used, along traditional materials, gabions, geocomposite materials, geocells, geotextiles, drainage pipes, geomembrane, special geotextiles for hidrotechnical work, geosenzor, mineral waterproofs, etc. In the process of using these new materials, is it extremely important to know their features and characteristics.

2.1. The treatment of foundation soil

For the construction of roads and dams, the geotechnical engineer deals with establishing the bonding between foundation soils and the areal.

The physical-mechanical features of soils imply several conditions regarding the site permeability and deformation in the particular case of the above mentioned structures. Following, in some cases must interfere for the improvement of site properties and assuring their impermeability. Changing the site permeability is determined mainly by changes in foundation soil porosity.
All these properties vary according to the rock type. As known, the rocks can be classified in three categories, magmatic, metamorphic and sedimentary.
In the case of stratification rocks the direction of water flow is influenced by the structural arrangements of mineral grains.
In the context of building a soil dam must be followed a basic condition of creating a perfect bound between the dam structure and the local foundation soil. In order to create the dam impermeability are being used different chemical and mechanical procedures, with the purpose to eliminate the existent faults.
The design of dam takes into consideration the local site conditions.

2.2. Foundations on permeable soils

In this soil category can be introduced sands, fine granular soils, silts.
Until the 20th century, the constructions on permeable foundation soils were realized using three main treatment methods in order to improve the site qualities. These methods were:

- Introduction of a impermeable vertical barrier,
- Realizing vertical retaining wall,
- Measures to control the water flow.

An interesting example, in the early 60s, is the research work activity of French engineers in soil dam construction process developed in French Alps. The foundation soil for the mentioned area contained sands and gravel with clays and layers of clays and marl.
Later on, the dam construction process was improved due to ”Solentache” method. The traditional mixture was composed of 20% clay, 13% Portland cement, 66% water for exterior layers and 25%clay, 5% cement, 70% water for inner layers. Along the technological development, the initial mixture has been diversified.

2.3. The method ”groove with paste”

The method “groove with paste“ was used for the very first time at the dam Wanapum in S.U.A. in 1958. After a wide series of researches and “in situ“ tests, it has been build a waterproofing barrier over the foundation width of 3,3 m at a maximum depth of 25 m with the method of groove with paste or mud.
The resulted mixture has been improved with a new composition containing soil-cement (Solentache method).

3. THE REDUCTION OF SUBTERANE WATER SEEPAGE

In case the construction of a waterproofing barrier for a dam realized on a permeable soil isn't viable, it is common to design a partial breakwater or cut-off. The ground water seepage can be diminished with one of the following methods:

- the construction of a vertical partial cut-off which usually extends downwards until it reaches un intermediary layer of soil with low permeability;
- the extension of the width at the base of the waterproofing section of the dam by constructing a horizontal layer of waterproofing soil extended upstream;

The two methods reduce the water pressure in downstream, increasing in this way the dam stability.
A complementary method was the design and construction of horizontal drainage.

4. SOLUBLE MATERIALS IN DAM FOUNDATION SOILS

The natural deposits of water soluble substances in foundation soil caused, for particular dams, several losses.
For example, in the Kern dam in California, the existence of gypsum in the foundation soil created discontinuities in its structure.
In the 1954 earthquake a large settlement of 75 cm was measured. The settlement was explained by the washing of great quantities of gypsum in foundation soil.
Another factor that might influence the dam behavior is determined by the earthquakes. This phenomenon determines changes in the site structure and texture.
One of the most important issues for dams designers is the stability analysis of the foundations on natural sand at liquefaction.

5. WATERPROOFING CONSTRUCTIONS OF DAMS

The waterproofing constructions of the dam structure are: screens, diaphragms and nucleus.
The waterproof parts of the dam will be connected with the sealing constructive elements of the foundation or directly with the foundation soil. It is recommended that the connection with foundation soil to be executed by a cut-off, positioned depth into the foundation soil. The dam screens must be strength enough to resist the loads applied, waterproofed, elastic and mobile. Based on these features is being determined the material to be used.

5.1 Screens of reinforced concrete

The screens made by reinforced concrete may be several kinds:

- stiff – support only deformation caused by temperature variation,
- sliding - supports both deformation given by temperature and settlement,
- elastic – supports large deformations.

A maximum height of the dams with concrete screens is about 120-140 m.

5.2 Screens made by asphaltic concrete

The asphaltic concrete was used for upstream protection of dams in the early of 20^{th} Century.

It was characterized by the following advantages:

- lower cost than concrete or steel,
- flexibility,
- easy to manufacture,
- the destroyed parts of seepage may be self-sealing by the slowly flow of bitumen.

The main disadvantages are represented by the fact that the material is not strong enough to protect the rocks fall and that incorporated bitumen is involved into an ageing phenomena.

The below figure 1 presents a dam with screens of asphaltic concrete.

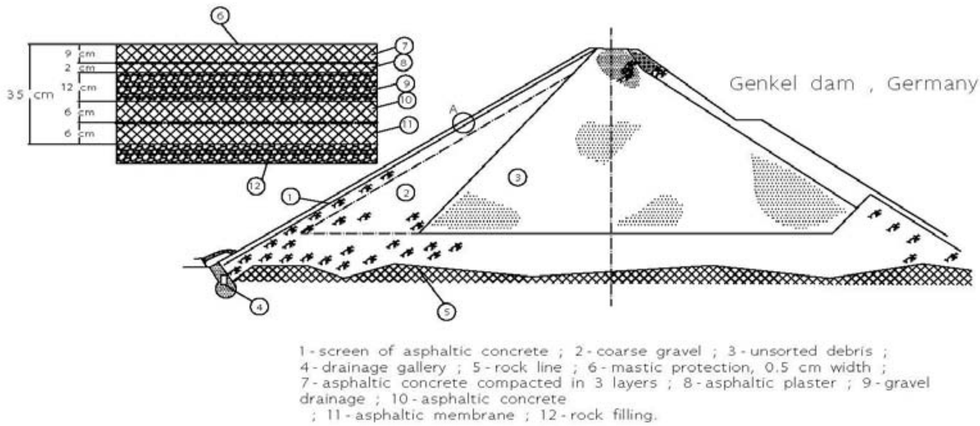

Figure 1. Genkel Dam, Germany [2]

5.3 Plastic screens

Among the first plastic materials used for dam protection were the polyethylene sheets and polymerized vinyl chloride sheets, with width of 0,10 - 1,50 mm layered one upon other. These materials have a good behavior while frosting – thawing process.

5.4 Metallic screens

The steel screens may be used in many situations. Although they are more expensive then reinforced concrete screens they present several important advantages:

- fully sealed,
- flexible,
- adaptability to different settlements without failure.

5.5 Timber screens

In the case of the total submersion of upstream dam frontage, the timber screens give the best results. Such screens were used successfully in Northern countries, for example Nissastrom Dam – Sweden; Aurajo Dam - Norway or Sabina Dam – USA.

6. CONCLUSIONS

The design and construction of natural dams enjoyed an extended usage in the 20th Century. Currently the modern materials and technologies have replaced the partly or totally the previous processes.
The present paper represents an introduction in the topic of natural dams construction, highlights also the main components of creation and protection of these constructions.

References

1. Bala, M, Baraje din materiale locale, Editura Tehnica Bucuresti, 1977,
2. Constantinescu, A.L, Tutan, I, Realizarile Institutului de Studii si Cercetari Hidrotehnice in domeniul urmaririi eforturilor si deformatiilor in fundatii si constructii de pamant, Studii de geotehnica, fundatii si constructii hidrotehnice, vol. III, Bucuresti, CSA 1967.

The Structural Response of the Cable Stayed Bridge of the Bucureşti – Braşov Highway at Bărcăneşti

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Summary

This paperwork presents the study, the results and the analysis of the structural response of a highway passage at Bărcăneşti - of the Bucureşti – Braşov Highway at the sector bucureşti – Ploieşti, the segment Moara Vlăsiei Ploieşti, km. 48 + 200 – 62+000 i.e. bridge over DN1at km. 60+425.

The structural system is of cable stayed bridge with two columns and a single plane of tie- rods placed in between the two lanes of the longitudinal axis of bridge (pair of beams). This is because of the required construction height and the architectural demand. The tie rod sistem is „harph". The bridge superstructure consists of 3 spans(40,5 + 80,0 + 40,5)m with a total length of 161,00m.

The bridge system was subjected to in situ tests. The tests were carried out in the static and the dynamic ranges with various convoys of 35t trucks in order to measure the structural response (strains, stresses, dynamic characteristics etc.).

In the end the experimental responses were compared to analytical calculus performed by the designer. All the test plan and the management was in accordance to the testing project of the bridge, advanced and assumed by the structural designer. This was to check the analytical assumptions and after that some conclusions were carried on.

KEYWORDS: *cable stayed bridge, static tests, dynamic in situ tests, structural response.*

1. INTRODUCTION

This paperwork presents the study, the results and the analysis of the structural response of a highway passage at Bărcăneşti - of the Bucureşti – Braşov Highway at the sector bucureşti – Ploieşti, the segment Moara Vlăsiei Ploieşti.

The passage from Bărcăneşti provides the crossing of the A3 highway Bucureşti – Ploieşti over the DN1 runway. The passage is sited at Km 60 + 425 on the highway and at Km 52 + 587 on DN1.

It is obvious that the passage is a very important construction and involves a special technology of rising up. It is also complex with respect to the prescriptions of STAS 12504 – 86. Under these circumstances it was necessary to develop a testing project by the designer of the bridge and according to that a series of static and dynamic tests to be carried on.

The tests provided important information regarding the strain state and the deflections measured in the characteristic cross sections and the stress state. The tests were necessary to control the correctness of the computational assumptions and also to monitor the structural behavior during the lifetime of the passage.

2. TECHNICAL FEATURES OF THE PASSAGE

On the crossing area the highway path is in a straight line with a 61 degrees angle oblique with respect to the over passed DN1. Due to this accentuated obliquity the necessary crossing span of the bridge resulted to be of 80m. The highway has a convex longitudinal profile with a 12300m radius.

In order to reduce the height of the construction and also because of aesthetical reasons it was selected a structural system type suspended bridge with two pilots and a single longitudinal plane of tie rods placed in between the two route lanes.

The cable system is “harp”. Due to the economical reasons a hybrid structure was selected; the external spans are made of reinforced concrete structures mixed with pre stressed concrete decks – these pre – stressed decks are extended 10m at both sides of the central span; the central span consists of a composite RC – steel structure, see Figure No. 2.

The superstructure consists of 3 spans (40,50 + 80,0 + 40,50)m with a total length of 161,00m. The transverse cross section of the bridge consists of two half cased beams with steel webs that work together with the reinforced concrete deck over a 60m length in the central span and two cased beams of pre-stressed concrete of 10m length on the rest of the central span and on side spans.

Figure 1. Passage over DN1 at Bărcăneşti – km 60 + 425. Total length: 171.20m

On the transverse direction the beams are connected with transverse beams made of steel or RC. The super structure height is 2,45m including the runway layers.

Figure 2. The 3 spans of 40,50m RC + 80,0m composite structure + 40,50 RC

In order to balance the forces developed in the structure on the side spans the super structure is built as cased sections made of pre stressed concrete. In this way the static model of the superstructure becomes the continuous beam with three spans and the connection points of the tie rods become the elastic supports for the beam.

The suspension rods are anchored to the superstructure by the means of a reinforced concrete transverse beam in the areas of the concrete deck (as depicted in the Figure No.3) and steel transverse beam (Fig. No. 4) in the area of the steel deck.

Figure 3. Anchorage of the harp in the RC transverse beams

By the means of the 12 transverse beams that are used for the anchorage of the ties is obtained also the overall spatial fixture of the two half deck of the superstructure, each of them corresponding to a traffic lane of the highway (2 lanes per direction) as it can be noticed from Figure No. 4.

The height of the superstructure deck (2.45m) leads to a slenderness of 1/32 when compared to the central (longest) span.

The infrastructure of the passage consists of two reinforced concrete columns and two RC lamellar abutments. The total height of the piers is 20,50m. The free height from the upper surface of the deck to the top of the pier is 15,00m.

The elevations of the columns are lamellar; they are sustained by a foundation plate of 1,50m thickness that is supported by 20 drilled piles of 1080mm diameter and 10,00m length. The piers have massive structure and are drilled each over 28 drilled piles of 1080mm diameter and 18m length, belted by the means of a 2,50m thickness foundation plate.

Figure 4. Anchorage of the harp in the steel transverse beams of the deck

3. THE STATIC TESTING PROGRAM

3.1. The static tests

In order to quantize the structural response in the static field (deflections and stresses) the bridge was loaded with convoys of 4 axle trucks with total load per truck of 350KN (± 7%) – Figure No. 5. The load was applied for all the spans of the bridge D1, D2 and D3 in the following patterns:

- Pattern 1: passage loaded with 28 trucks in 8 load cases (P1... P8);
- Pattern 2: passage loaded with 20 trucks in 8 load cases (P1'... P10').

The load schemes P1'... P10' can be superimposed and than obtain the schemes P1 ... P8, in the case that there are no available 28 trucks, but 20. The load positions are intended to lead to the most disadvantageous stresses and deflections on the lateral spans D1, D3 and central span D2.

In the testing project a number of 5 characteristic cross sections were considered:

- Section1: midspan of the edge span D1;
- Section 2: on the support at the column P1;
- Section 3: on the midspan of the central bay of the bridge, D2;
- Section 4: on the support corresponding to the column P2;
- Section 5: midspan of the lateral span D3.

Figure 5. Several 350KN 4 axle truck convoys

In those cross sections there were mounted flexi meters and geodesic transducers and markers. Also at that positions there were placed strain gages and inductive transducers (Fig. 6 and 7).

Figure 6 a. Positions for strain gages inside the RC caisson and b. inside the steel beam

Figure 7 a. Strain gages inside the RC caisson and b. the steel beam

3.2. The dynamic tests

The aims of the dynamic tests were the measurement of the dynamic parameters of the bridge, i.e. the natural frequencies, the fundamental period, the logarithmic decay of damping, the maximum deflections of the passage in the dynamic range when is subjected to the moving convoys with the patterns like those used in the static tests.

For the dynamic tests, several inputs were considered, like:

- Small amplitude seismic actions / time histories induced by truck passes over the measurement transducers - chains;
- Local shocks at the measurement points / cross sections; these were induced by impact of the truck over a wood block with 4 cm height placed at the midspan of the lateral bay D1 and at the central bay D2.

In the above figure it is depicted the transducers map (seismometers) used for the measurement input of the Bărcăneşti passage, according to the testing project.

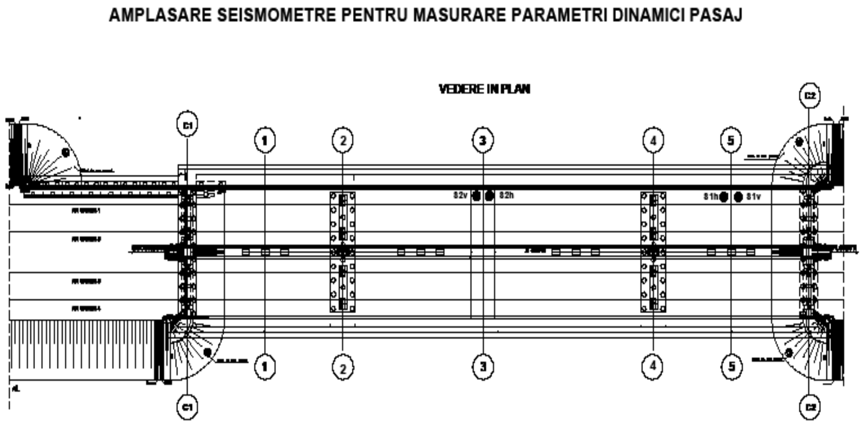

Figure 8 The transducers map

Again the maps were provided for flexi meters and inductive transducers in the case of other tests.

For the measurement of the dynamic parameters the data acquisition chains were mounted on the structure, based on seismometers placed at sections 3-3 and 5-5. The acquisition system is ESAM TRAVELER with 48 channels with dedicated computer software. The seismometers are KINEMETRICS RANGE SEISMOMETER with the 250v/g sensitivity.

4. TEST RESULTS

4.1. The static test results

In the Table no. 1 there are presented the deflections measured with the flexi meters in the lateral D1 span in the case of 20 trucks convoy, pattern number 2 and 10 load cases.

Table 1. Vertical deflections (mm) in the lateral D1 span

Fleximeters / Load patterns	**F1**	**F2**	**F3**	**F4**
1’	-4,80	-3,30	-2,40	-1,60
9’	-5,90	-3,90	-2,90	-2,00
2’	-7,60	-5,40	-3,90	-2,60
0	-0,50	-0,50	-0,10	-0,10
4’	-8,70	-9,30	-9,70	-9,30
3’	-7,10	-6,40	-6,90	-6,40
10’	-7,80	-7,50	-8,00	-7,40
0	+0,10	0,00	-0,10	-0,20
7’	+8,20	+6,90	+5,30	+4,60
5’	+8,00	+6,80	+5,50	+4,80
0	-0,10	-0,10	-0,20	-0,10
6’	+15,30	+15,00	+15,10	+14,60
8’	+15,70	+15,20	+15,30	+14,80
0	-1,40	-2,00	-2,00	-1,80

In the Table no. 2 there are emphasized the vertical deflections measured with geodesic geometric leveling in the load case no 1’ for bridge deck loaded with 20 trucks (pattern 2).

Table 2. Vertical deflections (mm) measured by geodesic leveling

Ipo-teza	Markers position leveling path no. 2									
	Zona stabila	Culee Ploiesti		40/2		Pila 1		80/2		Pila 2
	0	21	22	23	24	25	26	27	28	29
0	100.0000	99.9379	100.0821	99.9736	100.1344	100.0562	100.2208	100.0511	100.2238	99.8309
1	0	-0.2	-0.2	-4.3	-3.7	0.4	0.4	5.0	4.3	0.2
9	0	-0.4	-0.6	-5.7	-	-0.6	-0.2	4.9	4.0	-0.5
2	0	0.2	0.1	-0.3	-0.3	0.1	0.2	2.5	2.2	-0.6
0	0	0.0	-0.2	0.1	-0.1	-0.1	0.1	-0.1	-0.1	0.3
4	0	0.2	0.0	-0.8	-0.8	0.2	0.3	6.1	5.4	-0.2
3	0	-0.6	-0.8	-6.6	-6.7	-0.6	-0.4	7.2	7.0	-0.3
10	0	-0.4	-0.5	-7.5	-	-0.1	-0.1	7.8	7.2	-0.3
7	0	0.5	0.4	7.3	6.3	-0.4	-0.1	-36.8	-33.6	-1.3
5	0	0.7	0.4	9.6	8.1	0.1	0.4	-37.8	-	-1.2
6	0	1.3	1.0	17.1	16.8	0.7	0.5	-58.3	-	-1.1
8	0	1.1	1.1	14.8	14.4	0.4	0.5	-55.9	-58.1	-0.5
0	0	0.5	0.4	0.4	0.3	-0.2	0.0	-0.1	-0.4	-0.7

In the Table no. 3 there are depicted the stresses measured with resistive transducers in the load case no 1’ for bridge deck loaded with 20 trucks (pattern 2)

Table 3. Vertical deflections (mm) in the lateral D1 span

Load case no. 1’

Beam	Section	Position	E (MP)	ε (μm/m)	σ(MP)
Inside	5	Sup	34500	-6,160	-0,213
		Inf	34500	38,414	1,325
	4	Sup	34500	2,644	0,091
		Inf	34500	-29,336	-1,012
	3	Sup	210000	-10,016	-2,103
		Inf	210000	15,287	3,210
Outside	5	Sup	34500	-37,981	-1,310
		Inf	34500	35,082	1,210
	4	Sup	34500	12,740	0,440
		Inf	34500	2,090	0,072
	3	Sup	210000	-11,231	-2,359
		Inf	210000	5,632	1,183

Table no. 4 contains as example the strains measured in the characteristic sections measured with inductive transducers and calculated stresses for marginal spans

Table 4. Measured strains and calculated stresses

Ipoteza 8. 2 lanes 8 trucks on central span D2 + 2 trucks on Ploieşti abutment

Beam	Section	Position	E (MP)	Δl (mm)	ε (LVDT)	σ(MP)
Interior	1	Sup	34500	-0,028	-0,00003	-0,869
		Inf	34500	0,058	0,00005	1,821
	2	Sup	34500	-0,004	0,00000	-0,125
		Inf	34500	0,020	0,00002	0,627
	3	Sup	210000	-0,134	-0,00012	-25,525
		Inf	210000	-0,128	-0,00012	-24,475
Outside	1	Sup	34500	-0,003	0,00000	-0,103
		Inf	34500	0,114	0,00010	3,569
	2	Sup	34500	-0,053	-0,00005	-1,651
		Inf	34500	-0,003	0,00000	-0,092
	3	Sup	210000	-0,052	-0,00005	-10,015
		Inf	210000	0,099	0,00009	18,900

4.2. The dynamic test results

A large amount of digital data was processed regarding the natural characteristics of vibration as mentioned at pt. 3.2 and requested in the testing project. In the next figures there are depicted the Fourier Transform and the fundamental frequency of 1.086... 1.216 Hz and the response measured during time histories (convoy passes).

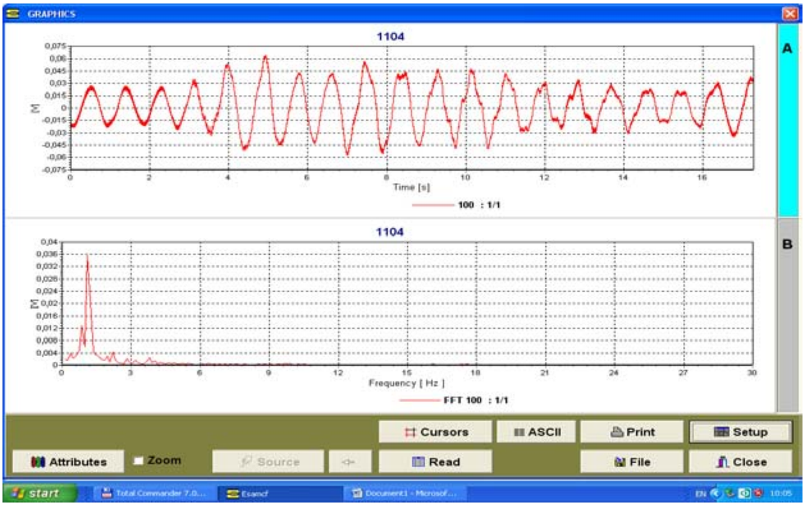

Figure 9 The Fourier Transform of the excitation at midspan D2

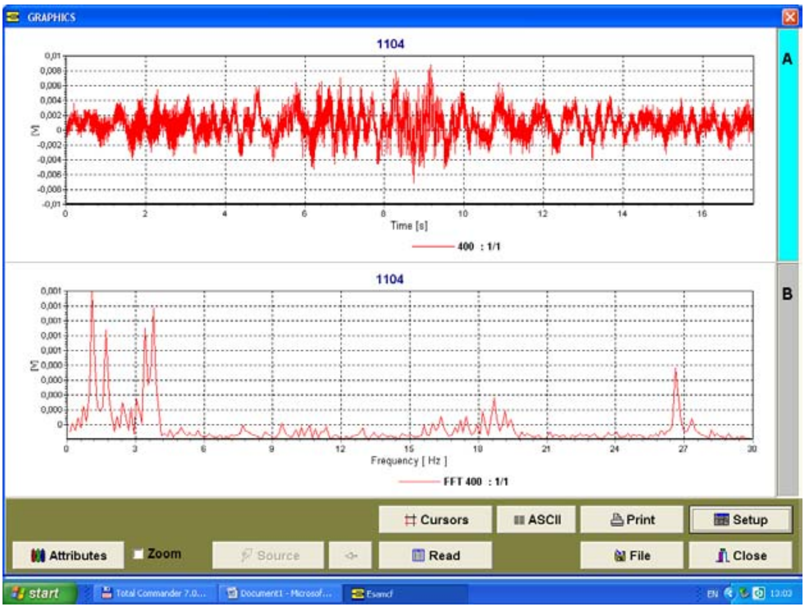

Figure 10 Time history analysis at midspan D2

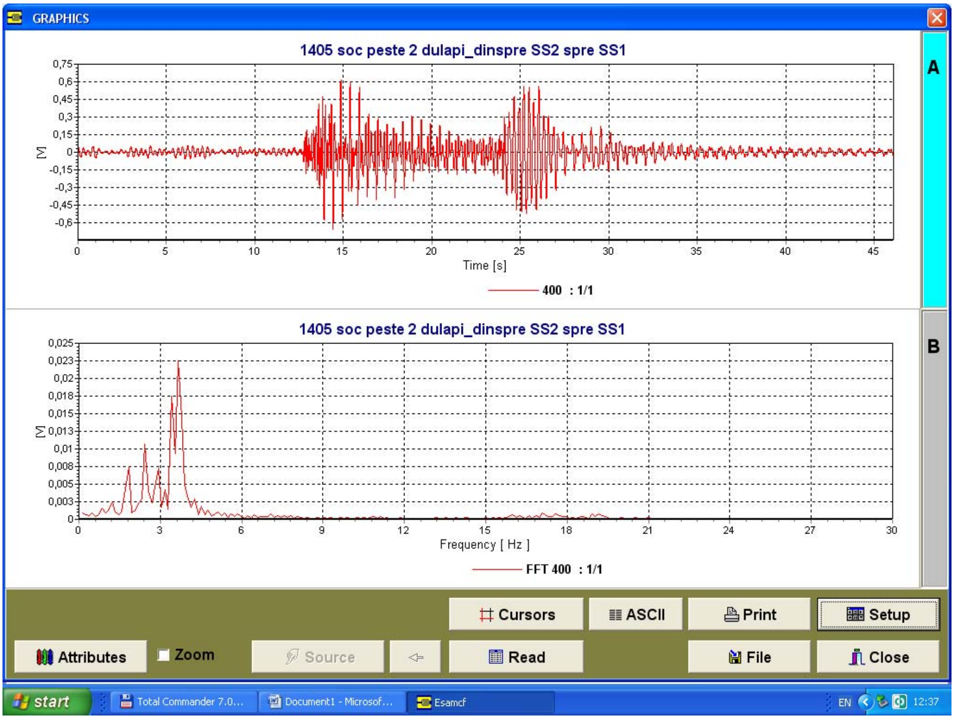

Figure 11 Impact type responses (2 timbers – path from SS2 towards SS1)

5. CONCLUSIONS

The live load used for the testing of the "Bărcăneşti" passage is greater than the live load stated in the Romanian Norm STAS 3221-65 ("E" loading class) and also greater than the load stipulated by SR EN 1991-2 (the computational convoy 1 – LM1).

The static deflection measured in situ for the outer beam at section 3 in the midspan D2 using the geodesic measurement was of 58,30mm when the loading convoy no. 6 acted. The calculated deflection is 55,10mm, thus the ratio of the measured/computed deflections is 1,059.

For the inner beam at section3 the calculated deflection is 58,1mm at the midspan D2, when the testing convoy no. 6 is used. The measured deflection is 55,10mm i.e. the ratio measured/computed deflections is 0,95.

In the case of the other sections it resulted a good correlation between the computed and measured vertical deflections; therefore it results a good behavior of the superstructure when is subjected to the loading convoys.

The measured vertical deflections are correct because two approaches were used – flexi meters and leveling and the values are sensitively equal.

The computational and measured stresses have the same magnitude for all the load cases.

The super structure of the passage didn't undertake any permanent deflection; this fact proves the righteous behavior in the elastic domain.

The deck is provided with righteous torsion stiffness; all the main beams achieved positive vertical deflections when convoys acted in eccentric loading positions (convoys on lane no.2).

The dynamic tests leaded to the fundamental frequencies in between 1.086... 1.216 Hz and a natural period of 0.94... 0.82sec.

References

1. Varlam, F. et. al., *Referat privind incercarea pasajului de pe "Autostrada Bucureşti – Braşov, sector Bucureşti – Ploieşti, sector 1B: MoaraVlăsiei – Ploieşti, sector C, km 48 + 200 – km 62 + 500"*, S.C. REXPOD SRL., 2012.
2. Budescu, M., *Noi concepţii privind protecţia seismică a structurilor*, Ed. Soc. Acad. "Matei – Teiu Botez, 2005.
3. Jantea C, Varlam, F., Comisu, C.C., *Poduri Metalice: Suprastructuri cu platelaj ortotrop (exemplu de calcul)*, Iaşi, Casa de Editura Venus, 2000.
4. Radu P.P., Negoescu E., *Poduri din beton armat, Probleme speciale*, vol. III, Institutul de Construcţii, Bucureşti, 1991.
5. Gutiu Şt. I., Păcurar V, Moga P., *Probeme speciale de poduri. Structuri compuse oţel – beton*, UTPRESS, Cluj - Napoca, 2007.
6. STAS 2920 - 83 *Poduri de şosea. Supraveghere generală şi revizii tehnice*.
7. STAS 2924 - 92 *Poduri de şosea. Gabarite*.
8. STAS 3221 - 90 *Poduri de şosea. Convoaie tip şi clase de încărcare*.

Study on statistical-probabilistic evaluation the quality level achieved by execution of an asphalt pavement

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Summary

This paper presents the results of quality evaluation an asphalt pavement realised on a representative sector within project construction of detour motorway of the city Sibiu. Laboratory data supplied by the Contractor have been analyzed, using statistical-probabilistic methods .Finally significant statistical parameters such as mean value, standard deviation, coefficient of variation, and coefficient of homogeneity are analysed. Based on these results the pavements characteristics, such as thickness of layer, density, compaction degree and water absorbtion are assessed.

KEYWORDS: quality evaluation, asphalt pavement, probabilistic methods, statistical parameters

1. INTRODUCTION

Road specialists are faced daily with a high volume of experimental data and information. This situation is caused both the diversity of new materials entering the road sector and by the multitude of experiments performed directly on the road network. Unlike other engineering constructions, the roads pavement condition depends on objective and subjective factors, thus becoming absolutely necessary, the use in the current practice, the statistical analysis of data.

The application the statistic methods for quality evaluation of building materials, technological processes and implicity of quality of the work carried out, permits to obtaine precise quantitative evaluation of the quality achieved , even when using a relative reduced number of data .

2. TECHNICAL DATA ON INVESTIGATED PAVEMENTS BINDER

The construction the BAD25 asphalt layer of the highway project was conducted during 2006, in accordance with the provisions of the Tehnical Specifications, established for this type of mix.
Laboratory and field data for the BAD25 mix such as asphalt layer thickness, apparent density of cores, degree of compaction and the water absorption have been recorded in the Annex 1.

3. INTERPRETATION OF THE DATA

For the purpose of assessing the quality of the execution of the binder layer in accordance with the Technical Specifications of this project, a total number of 96 values extracted from the Annex 1 have been statistically analyzed on specific forms, by considering following statistical indicators:

- average value of the string of data $\bar{x}$;
- dispersion D;
- standard Deviation σ;
- the coefficient of variation Cυ;
- the coefficient of homogeneity C_o.

Table 1. Critical values of the distribution of Student parameter t [1]

α / υ= n-1	0,200	0,100	0,050	0,025	0,010	0,005	0,001
1	3,078	6,314	12,706	25,452	63,657	127,32	-
2	1,886	2,920	4,303	6,205	9,925	14,089	31,598
3	1,638	2,853	3,182	4,176	5,811	7,453	1,911
4	1,538	2,132	2,776	3,495	4,604	5,598	8,610
5	1,476	2,015	2,571	3,163	4,032	4,773	6,859
6	1,440	1,942	2,447	2,969	3,707	4,317	5,959
7	1,415	1,895	2,365	2,841	3,499	4,029	5,405
8	1,397	1,860	2,306	2,752	3,355	3,832	5,041
9	1,383	1,833	2,262	2,685	3,250	3,690	4,781
10	1,372	1,812	2,228	2,634	3,169	3,581	4,587
11	1,363	1,796	2,201	2,593	3,106	3,497	4,437
12	1,356	1,782	2,179	2,560	3,055	3,428	4,318

13	1,350	1,771	2,160	2,533	3,012	3,372	4,221
14	1,345	1,761	2,145	2,510	2,977	3,326	4,140
15	1,341	1,753	2,131	2,490	2,947	3,286	4,073
16	1,337	1,746	2,120	2,473	2,921	3,252	4,015
17	1,333	1,740	2,110	2,458	2,888	3,222	3,965
18	1,330	1,734	2,101	2,445	2,878	3,197	3,922
19	1,328	1,729	2,093	2,433	2,861	3,174	3,883
20	1,325	1,725	2,086	2,423	2,845	3,153	3,850
21	1,323	1,721	2,080	2,414	2,831	3,135	3,819
22	1,321	1,717	2,074	2,406	2,819	3,119	3,792
23	1,319	1,714	2,069	2,398	2,807	3,104	3,767
24	2,318	1,711	2,064	2,391	2,797	3,090	3,745
25	1,316	1,708	2,060	2,385	2,787	3,078	3,725
30	1,310	1,697	2,042	2,360	2,750	3,030	3,646
40	1,303	1,684	2,021	2,329	2,704	2,971	3,551
60	1,296	1,671	2,000	2,299	2,660	2,915	3,460
120	1,289	1,658	1,980	2,270	2,617	2,860	3,373

The calculation of the values of the homogeneity coefficient C_o, has been conducted by considering the Student parameter t [1] in accordance with Table 1 function on the value of the degree of freedom ($\upsilon = n - 1$) and for a risk coefficient $\alpha = 0{,}05$ corresponding to a confidence level P = 95%.
The risk coefficient $\alpha = 0{,}05$ has been also used for the assessing the statistical value of population X_p, using the relation (1):

$$\overline{X}_e - \sigma * t/\sqrt{n} \leq X_p \leq \overline{X}_e + \sigma * t/\sqrt{n} \qquad (1)$$

The average values obtained for each parameter have been compared with the specified design values in the contained contract documents.
On the whole, all those results have been analysed and final conclusions on the degree of achievement of the designed level of quality for each characteristic have been drawn.
The following relations have been used for calculation of main statistical parameters $\overline{x}$, D, σ, C_υ, C_o,:

$$\overline{x} = \frac{\sum_{i=1}^{n} x_i}{n} \qquad (2)$$

where:

$\overline{x}$ - is the mean value of the string;
x_i - a individual value of the string
n - the number data (values) string.

$$\mathbf{D} = \frac{\sum_{i=1}^{n}(\bar{x} - x_i)^2}{n} \tag{3}$$

where:

D - is dispersion value of string
$\bar{x}$ - is the mean value of the string
x_i - a individual value of the string
n - the number data (values) string.

$$\boldsymbol{\sigma} = \sqrt{\frac{\sum_{i=1}^{n}(\bar{x} - x_i)^2}{n}} \tag{4}$$

where:

σ - is standard deviation of the string;
$\bar{x}$ - is the mean value of the string
x_i - a individual value of the string
n - the number data (values) string.

$$\mathbf{C_\upsilon} = \frac{\sigma}{\bar{x}} \cdot 100 \tag{5}$$

where:

Cυ - is the coefficient of variation;
σ - is standard deviation of the higher-level;
$\bar{x}$ - is the mean value of the string

$$\mathbf{C_o} = 1 - \frac{t \cdot C_\upsilon}{100} \tag{6}$$

where:

Co - is the coefficient of homogeneity;
Cυ - is the coefficient of variation;
t - parameter is Student (1.988).

Table 2. Results of statistical analyses conducted for the evaluation of quality level for the binder course

Crt No	The feature analyzed	The unit of measurement	The values of the statistical parameters					Average the value of population $X_e - \sigma * t \leq X_p \leq X_e + \sigma * t$	Design value	Comments
			$\bar{X}$	D	σ	C_v (%)	C_o (%)			
1.	Layer thickness	mm	62	19	4	7	0,86	$61 \leq X_p \leq 63$	62	Average value is in the designed range
2.	The density asphalt cores	g/cm^3	2,32	0,0	0,02	0,79	0,98	$2{,}312 \leq X_p \leq 2{,}319$	2,32	Average value is in the designed range
3.	The compaction degree	%	99,9	0,4	0,6	0,6	1,0	$99{,}7 \leq X_p \leq 100{,}0$	99,9	Average value is in the designed range
4.	The water absorption	%	4,2	0,5	0,7	16,6	0,7	$4{,}1 \leq X_p \leq 4{,}4$	4,2	Average value is in the designed range

The synthetic results of statistical analyses conducted for the evaluation of quality level achieved for the binder course on the site, are presented in Table 2.

4. CONCLUSIONS AND RECOMMENDATIONS

Based on the results presented in Table 2, on may conclude the following:

1. The average value of the layer thickness is varying between the limits 61 and respectively 63 mm, the interval variation being less than permitted interval: 54…66 mm.

Thus, it can be concluded that, the real work of construction of a binder course from a point a view of a thickness has been achieved with a better precision then that specificated in a contract.

2. This average value of density of the asphalt cores X_p is varying between 2,312 g/cm^3 and respectively 2,319 g/cm^3 being much higher than the designed value of 2,200 $g/cm^{3,}$ thus demostrating that the binder course has

a significant higher (5.22%) than the designed value, wittnising an excellent designed and construction work.

3. The value of the compaction degree is varying between 99,77 and respectively 100,00 as meeting the minimum designed value of 96%. This is wittnising an excellent compaction work achieved for the compaction of the binder course.

4. The average value for the water absorption varying between 4,1 and respectively 4,4 that meeting the designed values of the interval 3...8. This is wittnising a very good air volume for the binder course.

5. As a general conclusion, based on the statistic – probabilistic analyses conducted for the binder course on may affirme, that the quality level achieved for this layer is excellent.

Therefore, this study highlights the significant advantages of using the statistic – probabilistic approach for the quantitative evaluation of the quality of road works.

References

1. Andrei, Radu, *Metode statistice aplicate la drumuri, Editura Tehnica Bucuresti, 1983*
2. *Todini, Site Archive, Asphalt Work Laboratory Results, Project Sibiu Motorway.*

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