COMPUTATIONAL CIVIL ENGINEERING

C. Ionescu, F. Paulet-Crainiceanu, H. Barbat

editors

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II. Păuleţ-Crăiniceanu, Fideliu (ed.)
III. Barbat, Horia (ed.)

624

TABLE OF CONTENTS

DEVELOPMENTS AND TRENDS IN COMPUTATIONAL CIVIL ENGINEERING

A Review of Papers from the International Symposium "Computational Civil Engineering"
Iasi, Romania, October 24, 2003

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Introduction

Without any doubt, Computational Civil Engineering is a very wide and generous domain. However, to define this domain as an independent domain is very hard. This domain is mainly one of support and a development environment for the other Civil Engineering sub-domains. It is very probable that there is no Civil Engineering specialist that can escape the influence imposed by the use of computers and numerical methods specific to Civil Engineering.

Another important aspect of the Computational Civil Engineering is given by the general fact that the computational power is exponentially growing with time at comparatively same price. That leads to new perspectives and new findings through computational analysis and simulations and therefore to a highly dynamic field.

The above ideas are excellent shown by the works presented during the International Symposium "Computational Civil Engineering", [1]. In what follows it is intended to make a short review of the domain reflected by these works.

Developments and Trends

The field of Computational Civil Engineering is illustrated in [2] where a simple, sure and efficient method for seismic analysis of highway bridges is proposed. This method is based on the *flexible pier-rigid deck* hypotheses for bridges structures transversal seismically loaded. The paper concentrates on a local structural damage index based on damage constitutive loads applied on each point of the structure. After the complex mathematical algorithm is presented, a numerical example is using the Warth Bridge from Vienna, Austria as application structure. Fragility curves are obtained and shown.

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A computer program named *TreMuri* is used by Barbat, Oller, and Gómez Soberón, for determining the seismic fragility curves for a 60 years old six level masonry building typical for Barcelona, Spain, [3]. This computer program features a macro element model consisting of panels, simulating piers and architraves, connected by means of rigid blocks allowing simulation of walls with holes in a simple manner. Fragility curves for this kind of buildings will make possible to obtain damage scenarios for Barcelona.

In [4], an expert system based on fuzzy sets and neuronal networks is proposed by Bonett, Barbat, and Pujades. It is known that after major earthquakes, the number of building to be evaluated can be very large and the number of specialists well trained is very limited. Therefore the main goal of the proposed system is to assess the damage level of buildings after earthquakes even by neophytes. An artificial neural network is used to calibrate the expert system using the criteria established by specialists. Then the expert system can assess the damage level of buildings and, based on this, it is possible to take decision on the actions to be taken.

The wide area of the Computer Civil Engineering field is also reflected in the complex analysis described in [5]. The authors, Moreno, Barbat, and Pujades develop a methodology for assessing the seismic behavior of framed structures. First, a computer program, RUAUMOKO, is used for a non-linear static and dynamic analysis in 2D requiring to model the structure by means of several plane frames, connected each other. Then, to calculate the non-linear mechanical characteristics of the cross sections, BCSEC computer program is employed. STAC, a stochastic analysis computer program, performs Monte Carlo simulations. The distribution functions for the mechanical characteristics of the materials were defined and variation coefficients were assigned. The structural capacity is calculated by means of a non-linear static analysis with the RUAUMOKO program, by applying a monotonic increasing load to the structure until the ultimate load is reached. Finally, fragility curves are obtained.

Malcoci and Cociorva, [6], stress the importance of modeling with finite elements that lead to reduced costs in designing and building bridges structures. Numerical models together with experimental application show the way to obtain data about safety and resistance of a given bridge.

A solution for calculation and rehabilitation of a 100 year old bridge is presented by Boldus, B., Boldus D., and Bancila in [7]. The analysis was accomplished with FEM, utilizing MSC-NASTRAN program. Simulations were performed with PRESIM 98, an oriented computer program created at Technical University of Munich with the aim to simulate the effect of a truck's going over the bridge.

Chiotan, in [8], observes that the actual codes provide a simplified calculation of the reinforced concrete slabs from the bridges structures. This fact leads to the underestimation of the real interaction between the slabs and the bearing girders. Therefore, the paper is developing a numerical nonlinear analysis based on finite elements and correlates it with measurements and tests in situ. This leads to clarifications on the analysis and calculation models.

A study on establishing the curves of equal value of octahedral shear stress ratio for asphalt mixtures from wearing course is presented in [8]. The author, C. Racanel, bases the calculation on the tensions and deformations state obtained through tri-axial tests and also obtained through linear-elastic analysis using a computer program.

Rectangular hallow sections of cold form steel elements is the aim of the paper [10]. Finite element analysis and also experimental work are employed. It is shown that the costs of the experimental tests to fatigue-bending on the rectangular hallow sections can be significantly reduced by use of the finite element analysis especially when welding is necessary.

Kollo and Munteanu present a computer program of designing for stability the jointless railway submitted to temperature changing in [11]. This computer program is giving as results the safety coefficients. An application is showing the efficiency of the described algorithm.

Conforming to the European and Romanian seismic codes, Pacurar, Gutiu and Moga, [12], present the design of rehabilitation and modernization of the existing structure of a roof belonging to the Municipal Stadium in Cluj-Napoca City is presented. They aim of choosing the optimal shape for the structure, to induce a simple execution in the workshop, to obtain a minimum number of joints in-situ, and to achieve a safe exploitation and easy maintenance. Numerical results show the actual design data.

Differential nonlinear equations models are commonly used for research on bending stressed elements. Filipescu, Rusu, and Mihalache present, [13], an alternative to numerically solving these equations based on the use of wavelets Haar functions. A simple beam with one fix-end and one hinged-end vertically and uniformly loaded is used for validating the approach. Results obtained using Matlab are showing very precise and encourages future larger studies.

Barsan and Ignat present a series of four papers [14-17] that are a good example of use for Computational Civil Engineering. The first paper, [14], treats the problem of dimensioning the water-filtering layer of sand. Numerical solution of mass transfer equations is done through a computer program. Graphical results help understanding the phenomena and solve the problem. A second paper, [15], deals with a model of following the chlorine concentration evolution in water during the time of network transit. Solving this problem is a very important task and therefore the paper is a valuable contribution. A third problem, [16], involves the same problem of chlorine concentration but inside the water tanks. It is well known that inside water tanks complex phenomena take place. The paper shows an application of the proposed method for optimization of chlorine dosing in a inflow-outflow water tank. Another optimization problem is in the views of the fourth paper, [17], optimization of lamella settlers used in drinking water filtering with the aim of minimization of the critical settling velocity. The study shows that the smaller angle of inclination and the smaller distance between the plates the greater efficiency lamella settlers have.

In paper [18], Ionescu presents an integrated bridge design system. A 22 steps principle diagram describes in cybernetic concept the design, construction, exploitation, maintenance and technical-economical evaluation of bridges. This complex analysis is based on system theory and on similarities with mecatronics. The paper is highly original and opens a new angle of study the Bridge Engineering.

Based on Eurocode 3, Aanicai, [19], is using two plastic behavior of the material for continuous thin-walled beams. After a finite element analysis on a given beam, an interesting conclusion is withdrawn: ideally plastic model overestimates plastic rotation and underestimates ultimate load in dimensioning compared with the ones yielding from the softening moment resistance model, but the difference in design is not significant.

The Total Quality Management concept for the road administration is introduced by Dorneanu in [20]. Computerized procedures using databases for roads and bridges are created and employed. Screens shots from the software present these procedures. Conclusions are showing a decrease of the decision time for the problems related to the road traffic, the possibility for them to be located, removal of wrong data included in the data bank and the easiness to update the records.

The Structural Control domain is analyzed and evaluated by Pastia in [21]. A classification of the main types of control (passive, active, hybrid, semi-active) and the corresponding devices is revealed.

Luca, in [22], a parametric study is carried out to determine how the energy dissipation devices modify the structural response. The conclusion is that the use of energy dissipation devices coupled with steel braces allowed to stiffen the building, i.e. reducing the natural period, and to dissipate energy when the devices start yielding.

Doloca presents a new software tool *ENCALC* designed for the evaluation of the thermal energy consumption in buildings [23]. It is a user-friendly computer program for architects and civil engineers in various situations like: thermal rehabilitation of the old residential buildings, schools, hospitals, and new building design. The software is based on the energy balance in stationary conditions and takes into account the external temperature variations, the internal heat input and the heat coming from the sun radiation.

Budescu, Ciongradi and Roşca note in their paper, [24], that in certain situations, some earthquake engineering design codes provide increases of the computed seismic forces, in order to strengthen the serviceability safety under seismic loading, without any scientific reason. Therefore modeling problems (theoretical models, computational model, discretization) are largely and deeply commented. After that the analysis is focused on design steps, shown also graphically by a diagram. Between the conclusions it can be withdrawn that no matter how correctly and completely they are, the computational models cannot replace the general rules of structural conception. And also there is a suggestion regarding the ψ seismic code coefficient that could lead to a better appreciation in stresses.

Scinteie, in three papers, [25-27] is intensively using computers and computational methods. The first one, [25], deals with the concept of technical condition of a bridge which represents an evaluation of the entire set of characteristics of the bridge. Based on the concept of urgency an algorithm of bridge evaluation is presented. In [26], it is shown that the management of bridges implies a special attention in allocation of the funds where the technical and economical needs impose and where the benefits are maximal. Scinteie establishes priority orders in bridges management using a multi-criteria based index. The last paper, [27], deals with the problem of the shortest path, an essential problem in transportation. Its generalization, *k*-th shortest path, is vital for solving questions from ITS and dynamic traffic assignment. The paper describes the algorithm, the corresponding computer program and numerical applications.

Chitan and Strat, in two papers, [28] and [29], are mainly concerned with modeling problems. The first paper, [28], presents the choice of similitude scales and of adequate criteria in structural analysis in the case of dynamic loads, especially for investigations on vibrating tables. In the second paper, [29], there are briefly described some requirements of the dynamic similitude laws when modeling in a natural field of gravity and when using the same material for the model as for the prototype.

Conclusions

Papers presented at the International Symposium “Computational Civil Engineering” show the importance given to the computer work and also to the automatic calculation numerical methods in Civil Engineering. The diversity of topics and the complexity of the involved problems prove a deep sophistication and a more realistic analysis of problems in design and research. All these aspects have a strong support in computer science high level.

Based on new techniques and numerical methods there is an obvious tendency to accentuate the combination of deeper aspects from many sub-domains in Civil Engineering. A wide range of improvements in understanding the structural behavior, in easiness of design and increase of safety in Civil Engineering structures’ exploitations is shown.

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SIMPLIFIED MODEL FOR THE SEISMIC ANALYSIS OF HIGHWAY BRIDGES

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Abstract

A seismic vulnerability evaluation method based on structural analysis for RC highway bridges with simple pier bents is proposed in the paper. The proposed method is based on the hypothesis of the *flexible pier-rigid deck* behaviour of the structure subjected to transversal seismic loads. A flexible pier-rigid deck simplified model was therefore developed. This model has been chosen after verifying the correlation between the responses of the proposed model and of the real structure in its first few vibration frequencies. The study of the damage produced by the earthquake load is centered on the piers of the bridge, while the dynamic study of the deck can be performed after the structural analysis of the piers in an uncoupled way. The maximum damage of the piers under seismic actions is the principal aim of the proposed structural evaluation methodology. A local damage index is used for this purpose, which describes the state of the material at each point of the structure and is based on a constitutive damage law. The proposed model permits a simple, reliable and efficient structural analysis.

1. Introduction

Current methods for evaluating seismic damage to bridges can be divided into four main groups[1,2]: (1) obtaining of a vulnerability index by means of inspection; (2) evaluation of the damage through structural analysis; (3) estimation of vulnerability based on expert's judgment and (4) statistical analysis of actual data. The first method is based on simple evaluations aimed simply at providing a classification of those structures which show greatest seismic vulnerability. Models based on structural analysis provide a greater quantity of results, but reliability depends on their capacity to represent real seismic behavior. Evaluation based on expert's judgment requires a large number of professionals with in-depth knowledge of the problem and proven experience, while statistical evaluations based on real damage data can only be applied in zones of moderate or high seismicity where sufficient data are available.

This paper proposes a seismic vulnerability evaluation method based on structural analysis for RC highway bridges with simple pier bents. The proposed model is based on the characterization of the maximum damage of the piers of the bridge, the damaged inertia being obtained at their base. It is calibrated using experimental

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data and using an FE model. The application of the proposed model to the evaluation of fragility curves and damage probability matrices of a bridge built 30 years ago is finally given.

2. Simplified elastic model for reinforced concrete bridges

LC bridges with simple pier bents have greater redundancy and strength in their longitudinal direction; therefore greater damage will occur in piers when they are subjected to transversal excitation. Consequently, the proposed model is developed for the study of the response of bridges subjected to earthquakes acting in a transversal direction to that of the bridge axis. The simplified model shown in Figure 1 is based on the following hypotheses:

1. The piers are modeled as continuous elements with distributed mass and infinite axial stiffness.
2. The girders are modeled as perfectly stiff elements concentrating the mass at the top of the piers.
3. The bearings of the girders are modeled as equivalent short elements with circular cross section and real dimensions which work to shear.
4. The soil-structure interaction effect in piers and abutments is considered by means of linear springs that represent the rotational stiffness of the soil.
5. The abutments are considered to be perfectly stiff.

Accordingly, the transversal displacements at the top of the piers are the only degrees of freedom of the structural system.

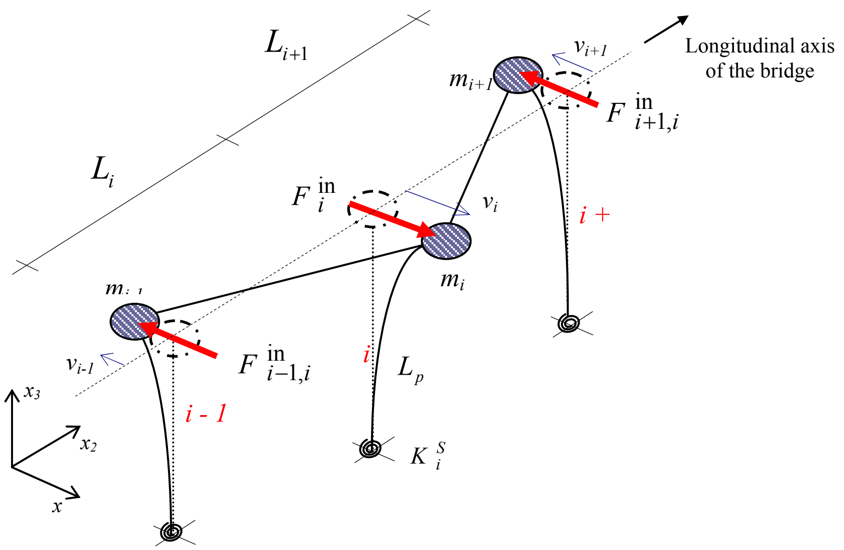

Figure 1. Basic scheme for the analysis of the bridge

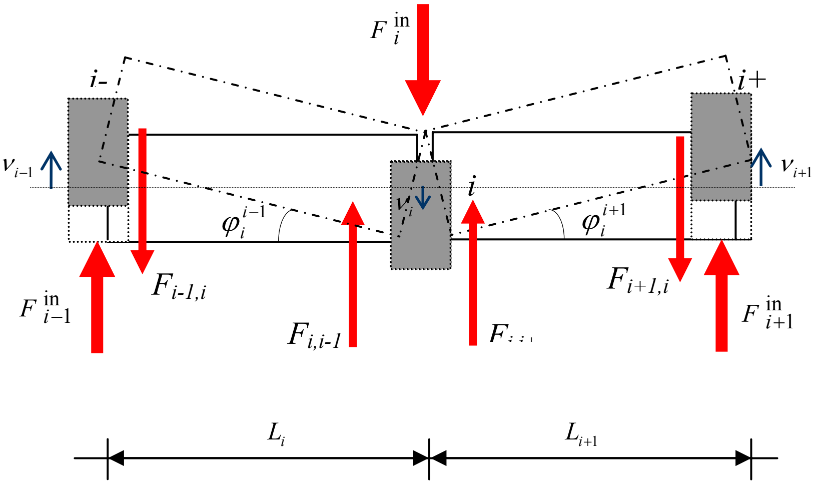

Figure 2. Girder rotation produced by the displacements at the top of the piers

The displacement of pier "i" generates the force distribution indicated in Figure 2, where F_i^{in} is the inertia force, $F_{i,i-1}$ and $F_{i,i+1}$ are the elastic forces produced by the rotation of the girders adjacent to pier *i*, and $F_{i-1,i}$ and $F_{i+1,i}$ are the elastic forces produced in the piers *i-1* and *i+1* respectively by the rotation of the contiguous girders. That is, for the pier under study, the sub-index indicates the pier on which the forces acted and the associated pier with it has the same girder. As it can be observed in Figure 2, the worst condition occurs when the two adjacent piers are displaced in the opposite direction to that of the displacement of pier *i*.

The bearings on each pier of the bridge are simulated as short columns with a circular cross section, whose behavior is governed mainly by shear deformations. The influence of the bearings on the behavior of the bridge is associated with their shear modulus, *G*, the distance in plan between the geometric centers of the bearings of each pier, h_a, their height, *a*, and the area of their cross section, A_p. On the basis of the above mentioned hypotheses, the total elastic force, R_i, due to the rotation of the adjacent girders to pier *i* is[2]

$$R_i = F_{i,i-1} + F_{i,i+1} = \left[\frac{G A_p h_a^2}{a L_i^2} + \frac{G A_p h_a^2}{a L_{i+1}^2}\right] v_i - \left[\frac{G A_p h_a^2}{a L_i^2}\right] v_{i-1} - \left[\frac{G A_p h_a^2}{a L_{i+1}^2}\right] v_{i+1} \quad (1)$$

where v_{i-1} and v_i are the maximum displacements at the top of the piers *i-1* and *i*. The inertia force at the top of the pier depending on the displacement v_i is obtained by means of the following equation[2]:

$$F_i^{\text{in}} = \frac{1}{\left[\dfrac{L_p^i}{K_i^S} + \dfrac{\left(L_p^i\right)^3}{3\,E_{c_i}\,I_i}\right]}\left[\begin{array}{l} v_i - \dfrac{\left[2\left(q_i^{\text{in}}\right)^{\max} + \left(q_i^{\text{in}}\right)^{\min}\right]\left(L_p^i\right)^2}{6\,K_i^S} \\ - \dfrac{\left[11\left(q_i^{\text{in}}\right)^{\max} + 4\left(q_i^{\text{in}}\right)^{\min}\right]\left(L_p^i\right)^4}{120\,E_{c_i}\,I_i} \end{array}\right] \tag{2}$$

where K_i^S is the equivalent stiffness of the soil, $\left(q_i^{\text{in}}\right)^{\max}$ and $\left(q_i^{\text{in}}\right)^{\min}$ are the maximum and minimum inertial loads by unit length produced by the horizontal acceleration, F_i^{in} is the total inertial force of the superstructure (girders) and L_p^i, E_{c_i} and I_i are the length, Young's modulus and the inertia of the cross section respectively.

At the top of each pier of the bridge, the total effective force is the sum of the forces R_i, produced by the rotation of the girders, given by Equation 1, with the forces F_i^{in} due to the displacement of the pier, given by Equation 2. By applying now Newton´s second law, the total effective force at the top of pier i is

$$F_i^T = R_i + F_i^{\text{in}} = m_i\,a_i \tag{3}$$

where m_i is the mass associated with the degree of freedom i, and a_i is the corresponding acceleration. Substituting the values of R_i and F_i^{in} into Equation 3, F_i^T is expressed as

$$\begin{aligned} F_i^T = m_i\,a_i = & \left\{\left[\frac{G\,A_p\,h_a^2}{a\,L_i^2} + \frac{G\,A_p\,h_a^2}{a\,L_{i+1}^2}\right] + \frac{1}{\left[\dfrac{L_p^i}{K_i^S} + \dfrac{\left(L_p^i\right)^3}{3\,E_{c_i}\,I_i}\right]}\right\} v_i \\ & - \left[\frac{G\,A_p\,h_a^2}{a\,L_i^2}\right] v_{i-1} - \left[\frac{G\,A_p\,h_a^2}{a\,L_{i+1}^2}\right] v_{i+1} \\ & - \left[\frac{1}{\left[\dfrac{L_p^i}{K_i^S} + \dfrac{\left(L_p^i\right)^3}{3\,E_{c_i}\,I_i}\right]}\right]\left[\begin{array}{l} \dfrac{\left[2\left(q_i^{\text{in}}\right)^{\max} + \left(q_i^{\text{in}}\right)^{\min}\right]\left(L_p^i\right)^2}{6\,K_i^S} \\ + \dfrac{\left[11\left(q_i^{\text{in}}\right)^{\max} + 4\left(q_i^{\text{in}}\right)^{\min}\right]\left(L_p^i\right)^4}{120\,E_{c_i}\,I_i} \end{array}\right] \end{aligned} \tag{4}$$

Defining from here the stiffness terms

$$K_{i,i} = \left[\frac{G\,A_p\,h_a^2}{a\,L_i^2} + \frac{G\,A_p\,h_a^2}{a\,L_{i+1}^2}\right] + \frac{1}{\left[\dfrac{L_p^i}{K_i^S} + \dfrac{\left(L_p^i\right)^3}{3\,E_{c_i}\,I_i}\right]} \tag{5}$$

$$K_{i,i-1} = \left[\frac{G\,A_p\,h_a^2}{a\,L_i^2}\right] \tag{6}$$

$$K_{i,i+1} = \left[\frac{G\,A_p\,h_a^2}{a\,L_{i+1}^2}\right] \tag{7}$$

and also the force

$$F_i^q = \left[\frac{1}{\left[\dfrac{L_p^i}{K_i^S} + \dfrac{\left(L_p^i\right)^3}{3\,E_{c_i}\,I_i}\right]}\right]\left[\frac{\left[2\left(q_i^{\mathrm{in}}\right)^{\max} + \left(q_i^{\mathrm{in}}\right)^{\min}\right]\left(L_p^i\right)^2}{6\,K_i^S} + \frac{\left[11\left(q_i^{\mathrm{in}}\right)^{\max} + 4\left(q_i^{\mathrm{in}}\right)^{\min}\right]\left(L_p^i\right)^4}{120\,E_{c_i}\,I_i}\right] \tag{8}$$

the final equilibrium equation can be written for each pier as

$$F_i^q + m_i\,a_i = K_{i,i}\,v_i - K_{i,i-1}\,v_{i-1} - K_{i,i+1}\,v_{i+1} \tag{9}$$

Applying this equation to each degree of freedom of the structure, a system of equations $\boldsymbol{F} = \boldsymbol{K}\,\boldsymbol{v}$ is obtained, where $\boldsymbol{K}$ is a tri-diagonal stiffness matrix, $\boldsymbol{F}$ is the force vector and $\boldsymbol{v}$ the displacement vector. As the transversal displacements at the girder-abutment connection are neglected, the final stiffness matrix of the bridge is of size *(n -2×(n-2)*, being *n* the number of piers and abutments of the bridge.

3. Non-linear formulation of the model

In elastic conditions, the solution of Equation 9 assures equilibrium at each time instant. However, when the non-linear behavior of the structural materials is taken into account, the equation of motion for each pier is written as

$$F_i^q + m_i\,a_i = K_{i,i}\,v_i - K_{i,i-1}\,v_{i-1} - K_{i,i+1}\,v_{i+1} - F_i^R \tag{10}$$

where F_i^R is the residual force. This unbalanced force is due to the fact that the stiffness coefficients $K_{i,i}$, $K_{i,i-1}$ and $K_{i,i+1}$ are not constant and consequently the solution of Equation 10 should be obtained through an iterative process.

To obtain the maximum damage for the bridge piers using the model described in Figure 1, the non-linear Equation 10 is solved using Newark's algorithm. In this analysis the balance condition is achieved by eliminating F_i^R by means of a Newton-Raphson process, which indirectly eliminates the residual bending moment, ΔM, which is the difference between the maximum external moment, M_e, and the internal capacity, M_{int}. For each step of the non-linear analysis the properties of the system are updated, considering the degradation of the material caused by the seismic action.

In the following the evaluation of the damage in the direction perpendicular to the bridge axis will be developed for any of the piers of the bridge, without a sub-index being written for the pier. Knowing the maximum displacement of a pier, the resultant force at the top and the maximum external moment at the base (predictor moment) can be obtained by means of

$$F_e = v\, k \tag{11}$$

$$M_e = F_e\, L_p \tag{12}$$

where

$$k = \frac{3 E_c I}{L_p^3} \tag{13}$$

is the initial bending stiffness of the pier, F_e is the elastic force produced by any external action at the top of the pier, M_e is the maximum external moment, v is the maximum displacement of the pier (obtained by means of Newark's algorithm) and E_c, I and L_p are the Young's modulus of the concrete, the inertia of the cross section and the length of the pier respectively. On the basis of the maximum external moment, it is possible to calculate the maximum damage that a pier can suffer due to seismic action.

In the case of seismic loads acting in the transversal direction of the bridge (x_1 axis), the elastic state of stress and strain in the longitudinal direction of the pier is

$$\begin{cases} \varepsilon(x_1, x_3) = \chi_1(x_3) \cdot x_1 \\ \sigma(x_1, x_3) = E_c \cdot \varepsilon(x_1, x_3) \end{cases} \tag{14}$$

where

$$\chi_1(x_3) = \frac{M_e(x_3)}{E_c\, I} \tag{15}$$

is the curvature of the pier, σ(·) and ε(·) are the stress and strain values, x_1 is the distance (in the direction of this axis) from the current point to the neutral axis of the cross section of the pier (see Figure 3), E_c is the initial Young’s modulus of the pier, and M_e is the maximum external moment acting on the element.

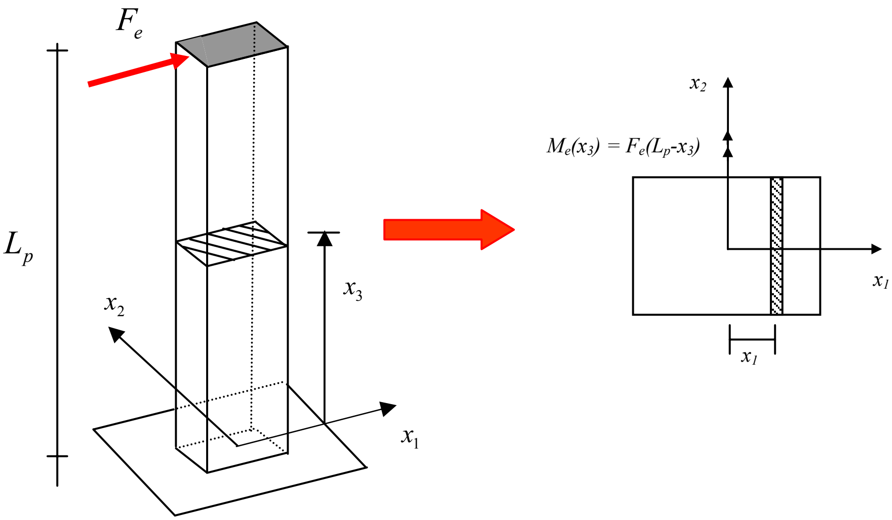

Figure 3. Simplified model of a pier used in the non-linear analysis

Substituting Equation 15 into Equation 14, the cross sectional states of stress and strain in direction x_1 are defined by

$$\begin{cases} \varepsilon(x_1, x_3) = \dfrac{M_e(x_3)}{E_c\, I} \cdot x_1 \\ \sigma(x_1, x_3) = \dfrac{M_e(x_3)}{I} \cdot x_1 \end{cases} \tag{16}$$

On the basis of these equations, the internal moment for the cross section of a pier is given by

$$M_{\text{int}}(x_3) = \int_{A_c} \sigma\, x_1\, dA_c \tag{17}$$

where the internal moment in the longitudinal direction, $M_{\text{int}}(x_3)$, is obtained by integrating the moments of the elemental forces $\sigma\, dA_c$ on the cross sectional area, A_c, of the pier.

When the pier remains within the elastic range, its internal and external bending moments are equal. However, when the yield limit of the material has been exceeded, the demanded moment, M_e, is greater than the resisting moment, M_{int}, and the residual moment is

$$\Delta M(x_3) = M_e(x_3) - M_{\text{int}}(x_3) \tag{18}$$

which should be less than an imposed tolerance.

When the pier suffers damage, the state of stress developed in the damaged cross section of the structure (Equation 16) is evaluated by means of the following equation:

$$\sigma(x_1, x_3) = f(x_1, x_3)\, E_c^d\, \chi_1(x_3)\, x_1 \tag{19}$$

where

$$E_c^d = f(x_1, x_3)\, E_c^0 \tag{20}$$

is the Young's modulus of the damaged material, E_c^0 is the undamaged Young's modulus, and $f(x_1,x_3)$ is the damage function, which will be defined later. Substituting Equation 19 into 17, the internal moment of the cross section of the pier is

$$M_{\text{int}}(x_3) = E_c^0\, \chi_1(x_3)\, I^d(x_3) \tag{21}$$

where

$$I^d(x_3) = \int_{A_c} f(x_1, x_3) \cdot x_1^2\, dA_c \tag{22}$$

is the inertia of the damaged cross section of the pier respecting the new neutral axis. For each time increment, a predictor moment is defined by means of the following equation:

$$M^0(x_3) = E_c^0\, I(x_3)\, \chi_1(x_3) \tag{23}$$

in which the elastic properties of the material have been used.

For each time increment in which the predictor moment produces an unbalanced load increment greater than a tolerance (Equation 18), the procedure considers an increment in the curvature in order to obtain a corrector moment which permits it to reach the equilibrium state. The convergence criterion used states that the stable response is obtained for the whole structure if

$$C_c = \sqrt{\frac{\sum_i \Delta M_i^2}{\sum_i \left(M_e^i\right)^2}} \leq TOL \tag{24}$$

where *TOL* is the tolerance adopted (*TOL*$\rightarrow$ *0)*.

The structural damage can be characterized at a given point and it is a reliable method of estimating the damage accumulation caused by a local micro-structural degradation obtained from the Continuum Mechanics[3-5]. In order to define the inertia and the internal moment of the damaged cross section of a pier, the isotropic damage model of Oliver et al.[5] has been applied. According to this model, the level of damage of the cross section of a pier is evaluated by means of the following damage function:

$$f(x_1, x_3) = 1 - d(x_1, x_3) \tag{25}$$

where

$$d(x_1, x_3) = 1 - \frac{\tau^*}{\tau(x_1, x_3)} \exp\left[A\left(1 - \frac{\tau(x_1, x_3)}{\tau^*}\right)\right] \tag{26}$$

where τ is the current effective stress, τ^* the effective stress threshold, and A a parameter depending on the fracture energy. The inertia tensor of the damaged cross section is calculated by means of a numerical algorithm.

Once the convergence of the process is reached and the damage is calculated at each integration point, the maximum damage at the base cross section of a pier can be obtained. Two pier damage indices and three global damage indices are used in this paper. The first pier damage index characterizes the maximum damage at the base of each pier of the bridge

$$D = \frac{M_e(x_3) - M_{\text{int}}(x_3)}{M_e(x_3)} \qquad \text{for } x_3 = 0 \tag{27}$$

Using the pier damage index 27 the global structural damage caused by seismic action in the bridge is described, a global mean damage index is defined as the average of the pier damage indices

$$D_m = \frac{\sum_i D_i}{n_p} \quad i = 1,\ldots,n_p \tag{28}$$

where n_p is the number of piers of the bridge.

4. Numerical example

The seismic behaviour of the Warth Bridge was analyzed as an example of the application of the proposed model. This structure, located 63 km from Vienna, Austria, was built 30 years ago. It was designed for a horizontal acceleration of 0.04g, using a quasi-static method. Now, according to the current Austrian seismic code, it is necessary to consider horizontal design accelerations of the order of 0.1g for this bridge site[6]. The Warth Bridge has two spans of 62.0 m and five of 67.0 m, with a total length of 459.0 m. The seven spans of the bridge give rise to six piers with heights of 31.0 m, 39.0 m, 37.0 m, 36.0 m, 30.0 m and 17.6 m, as can be observed in Figure 4.

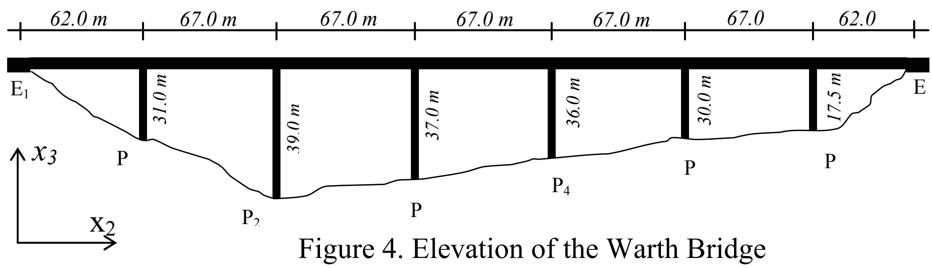

Figure 4. Elevation of the Warth Bridge

The statistical relationship between the input and output variables of the problem was obtained using Monte Carlo simulation. The probabilistic characteristics of the input random variables were defined using experimental and analytical tests performed in different researches[7-9]. A marginal distribution function and a correlation coefficient were assigned to each input variable. Given these values, the statistics of the output variables, in particular the principal statistical moments, histograms and accumulated frequency curves, were determined.

The theoretical fragility curves of the global damage index D_m, (Equation 28) of the Warth Bridge are shown in Figure 5, in which the curves associated with peak ground accelerations of 0.05 g and 0.10 g are not shown due to the fact that for these earthquake sizes the structure has a linear behavior. For all the considered peak ground accelerations, the Gamma theoretical distribution was fitted to the global damage index D_m.

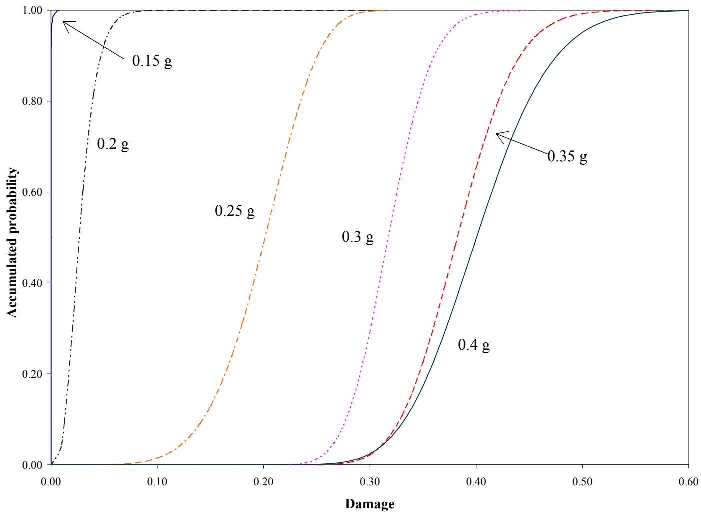

Figure 5. Fragility curves for the D_m global damage index

5. Summary and conclusions

In this work a model for the evaluation of the damage caused by seismic actions in RC bridges with single pier bents is developed. For this structure, the proposed model considers only one degree of freedom, namely the transversal displacements, at the top each pier of the bridge.

The damage caused by the seismic action is defined by applying an isotropic damage model based on continuum damage mechanics. Using this model, the inertia of the damaged cross section at the base of each bridge pier is obtained. A pier damage index and a global damage index of the bridge are used to characterize the seismic behavior of the structure.

The non-linear proposed model suitably calculates the maximum damages in the piers of RC bridges and it is a low-cost computer tool, ideal for the multi-analysis processes required by the evaluation of seismic vulnerability. The proposed analysis procedure was applied to determine the seismic vulnerability of the Warth Bridge by means of fragility curves using Monte Carlo simulations.

Acknowledgments

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SEISMIC FRAGILITY CURVES FOR TRADITIONAL UNREINFORCED MASONRY BUILDINGS OF BARCELONA, SPAIN

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Abstract

Residential buildings in Barcelona are mainly reinforced concrete buildings and unreinforced masonry buildings with particular features, typical of the customary constructive techniques of the city. The unreinforced masonry buildings of Barcelona have an average age of more than 60 years and were built without any consideration of the seismic hazard. The building used here as a typical example has six stories, 18.9 m by 24.5 m in floor and 24 m in height. For representing this typology, a six-story URM building with 18.9 m by 24.5 m in floor and 24 m in height has been selected.

To analyze the dynamic behaviour of the building we used *TreMuri* computer program (Galasco et al., 2002), which is an excellent tool to describe the non-linear in-plane mechanical behaviour of masonry panels and to assess the expected damage on masonry buildings due to earthquakes. The macro element model consists of panels, simulating piers and architraves, connected by means of rigid blocks. The macro element takes into account both the overturning mechanisms, related to cracking at the corners, and shear mechanisms by means of a damage model with friction. In this way a generic wall with openings can be described by means of a limited number of unknowns, involving few model parameters.

By using the *TreMuri* program, a push–over analysis was carried out by using a lateral loading pattern corresponding to the third mode of vibration of the structures. In this way, we obtained the capacity curve for the building and then, by using also the demand spectra the performance point of the structure was obtained. Considering a lognormal probability distribution, we finally obtained seismic fragility curves for the building. These curves will be used in developing seismic damage scenarios for Barcelona.

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1. Introduction

Residential buildings in Barcelona are mainly reinforced concrete buildings and unreinforced masonry buildings. The most of these buildings have been designed without any seismic consideration. An emblematic zone of the Barcelona city is the "Eixample". This sector of the city, was constructed according to the design of famous engineer Ildefonso Cerdá. The final project was approved in 1860. The Eixample is formed by numerous squares almost totally symmetrical of approximately 113×113 m. These squares are perfectly aligned, beveled in its vertices by means of edges of about 20 meters. The sector covers approximately 750 hectares of the surface of the city [4]. The unreinforced masonry buildings are the typical typology of the Eixample zone. Therefore, for representing this typology, an existing six-story building with 18.9 m by 24.5 m in floor and 24 m in height has been selected. In this work, the seismic performance and vulnerability of this building have been evaluated by using N2 method proposed by Fajfar[1] and fragility curves respectively.

2. Selected structural typology

The building selected for this work is localized in the "Eixample" district in Barcelona. This structure was built in 1882 without any consideration of earthquake resistant design. The building has a six-story unreinforced masonry structure. A typical floor plan and elevation are show in Figures 1. The plan is 18.9 m by 24.5 m, and the story height is variable.

3. Computer Model

To analyze the dynamic behaviour of the building the *TreMuri* program has been used, which is an excellent tool to describe the non-linear in-plane mechanical behaviour of masonry panels and to assess the expected damage on masonry buildings due to earthquakes[2]. The macro element model consists of panels, simulating piers and architraves, connected by means of rigid blocks. It takes into account both the overturning mechanisms, related to cracking at the corners, and shear mechanisms by means of a damage model with friction. In this way a generic wall with openings can be described by means of a limited number of unknowns, involving a few model parameters. These models have been verified through a comparison with the results of an experimental test performed on a building of Pavia, Italy. The model has been defined by using 8 walls in the x-direction, and 6 walls in the y-direction. Figure 2 shows the 3D model.

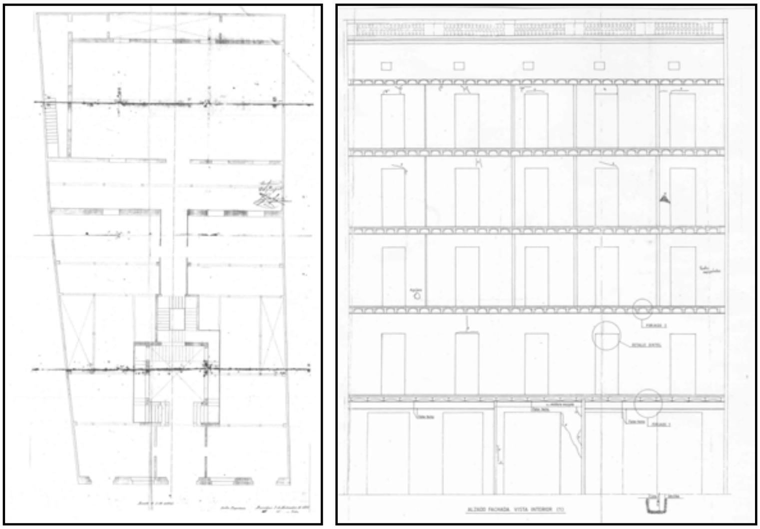

Figure 1. Plan and elevation views of the building.

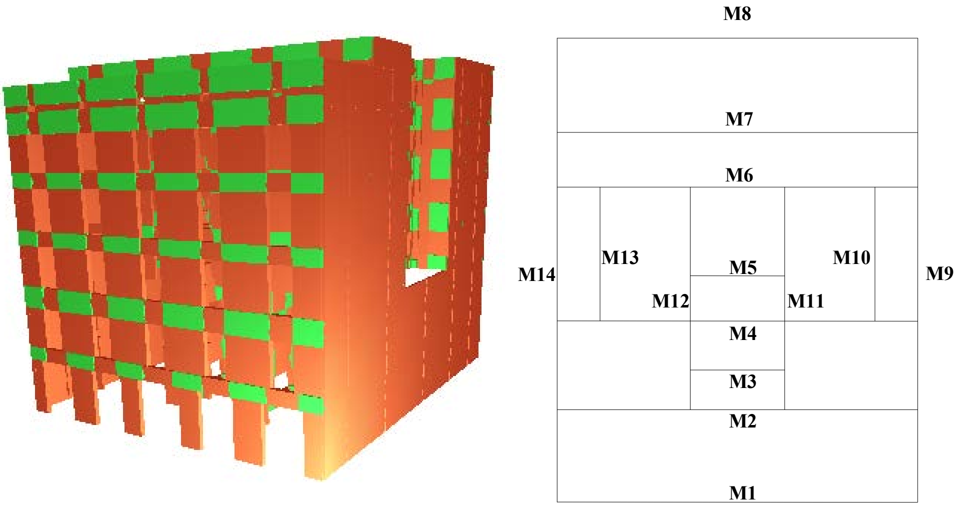

Figure 2. 3D model for the studied 6 story unreinforced masonry building.

4. Seismic performance evaluation

The N2 method proposed by Fajfar[1] formulated frame of the capacity spectrum method has been used with the aim of obtaining the performance point. This method requires starting from the definition of the demand and capacity spectra. The capacity spectrum is obtained by means of a pushover analysis performed by using the TreMuri program. The seismic demand which has been used corresponds to the deterministic analytical formulation for the acceleration response spectra proposed in reference [3].

Seismic demand

The value of the spectral acceleration is defined by mean of the following equations:

$$Sa = \begin{cases} PGA\left[1+\frac{T}{T_B}\left(B_C-1\right)\right] & 0 \le T \le T_B \\ PGA * B_C & T_B < T \le T_C \\ PGA*\left[\frac{T_C}{T}\right]^d * B_C & T_C < T \le T_D \\ PGA*\left[\frac{T_D}{T}\right]^2 * B_C & T > T_D \end{cases} \tag{1}$$

PGA is the peak ground acceleration, T is the period, B_C factor defined as $S_{a,\max}/PGA$, T_B, T_C are the limits of the constant spectral acceleration range, T_D is the beginning of the constant displacement response range, B_D is the factor defined as $S_a(T_D)/PGA$ and d is the variable exponent defined as follows:

$$d = -\frac{\log\left(\frac{\mathrm{B}_D}{\mathrm{B}_C}\right)}{\log\left(\frac{T_D}{T_C}\right)} \tag{2}$$

Figure 3 shows the response spectra proposed and the selected parameters for Barcelona.

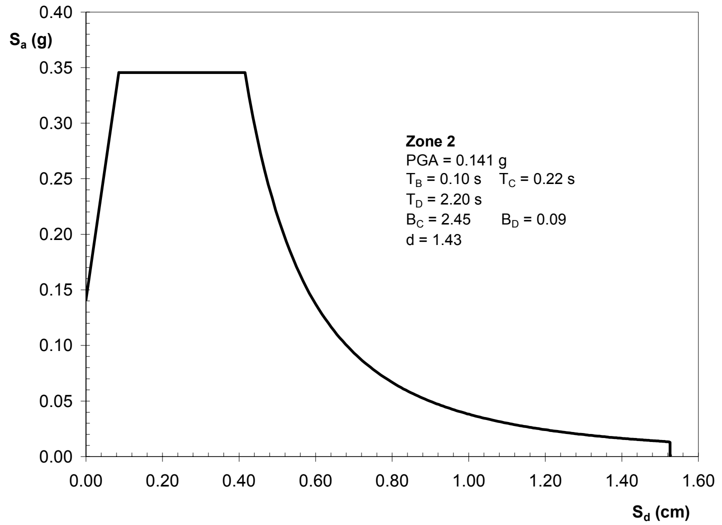

Figure 3. Deterministic response spectrum[3] used in this study.

4.1 Capacity spectrum

A push–over analysis has been carried out by means of the *TreMuri* program, using a lateral loading pattern which corresponds to the third mode response of the structures. In this way, the capacity curve of the building has been obtained. This curve is the plot of the static–equivalent base shear versus the roof displacement of the building. The base shear axis is then converted to spectral acceleration and the displacement axis is converted to spectral displacement. In this way the capacity spectra are easily obtained. Figure 4 shows their bi-linear representation.

4.2 Performance point

Figure 5 shows the graphic representation of the performance point for the studied building. It can be seen that the capacity spectrum cuts the deterministic response spectrum within the elastic range. Therefore, there is not reduction for ductility, and the structural response for this seismic demand remains within the elastic range.

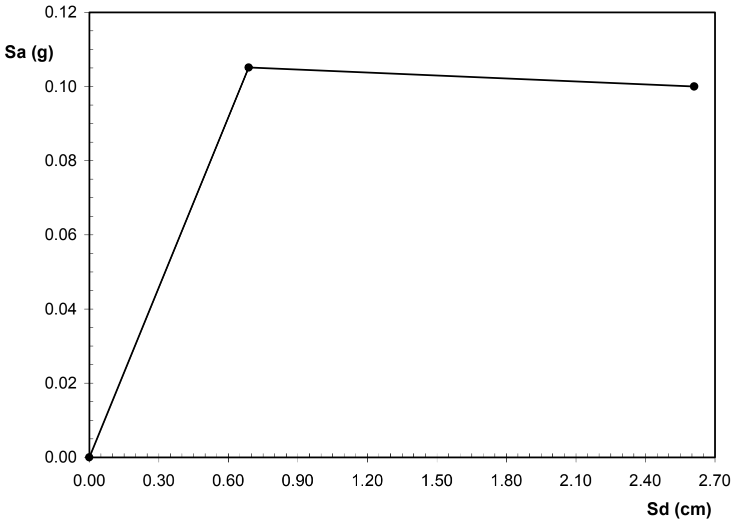

Figure 4. Bi-linear representation of the capacity spectrum.

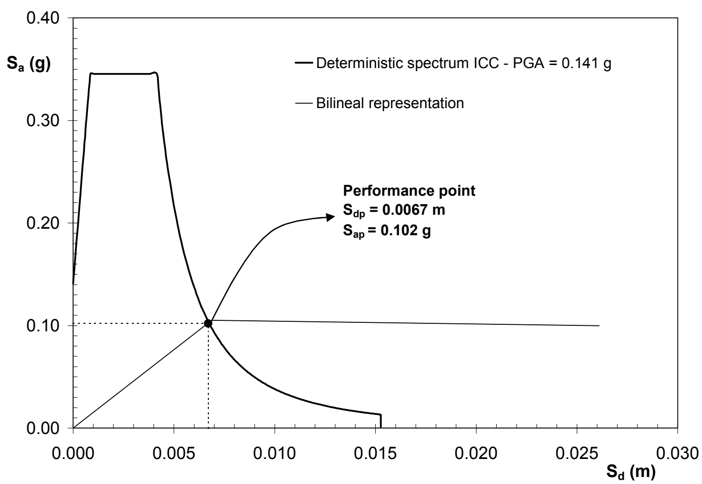

Figure 5. Demand and capacity spectra and the performance point.

5. Fragility curves

A deterministic analytical method proposed by the vulnerability working group of the RISK-UE project[4] has been used for evaluating the seismic vulnerability of the building. To generate the fragility curves, this method assumes that the probability of being in, or of exceeding a given damage state, corresponds to a lognormal distribution. Therefore, given a spectral displacement *Sd* and a damage state *ds*, this probability can be obtained from the following equation:

$$P[ds / Sd] = \Phi\left[\frac{1}{\beta_{ds}} \ln\left(\frac{Sd}{\overline{Sd}_{ds}}\right)\right] \tag{3}$$

$\overline{Sd}_{ds}$ is the mean value of the spectral displacement at which the building reaches the threshold of the damage state ds, β_{ds} is the standard deviation of the natural logarithm of this spectral displacement and Φ is the standard normal cumulative distribution function. Figure 6 presents the curves for all the damage states (no damage, slight, moderate, extensive and complete).

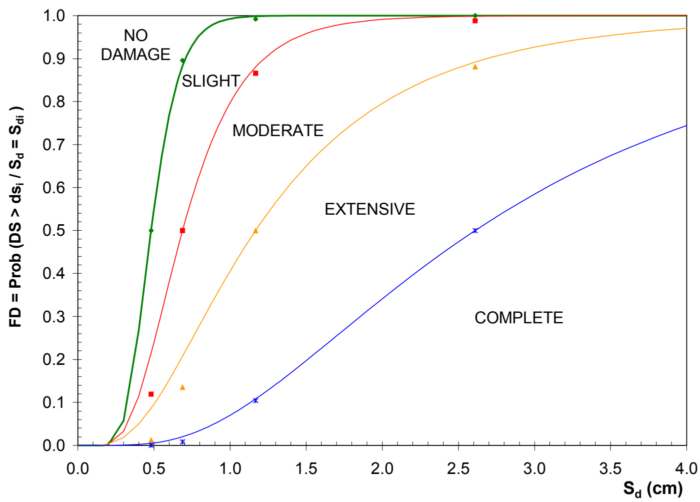

Figure 6. Fragility curves of the building.

Figure 7 shows the damage probability of each damage state corresponding to the value of displacement spectral demand (performance point) obtained for the unreinforced masonry building. The main structural damage can be classified as slight (40% approximately). However, the moderate and extensive damage have considerable damage probability (28% and 18% respectively). In general, the seismic performance corresponds to the operational level.

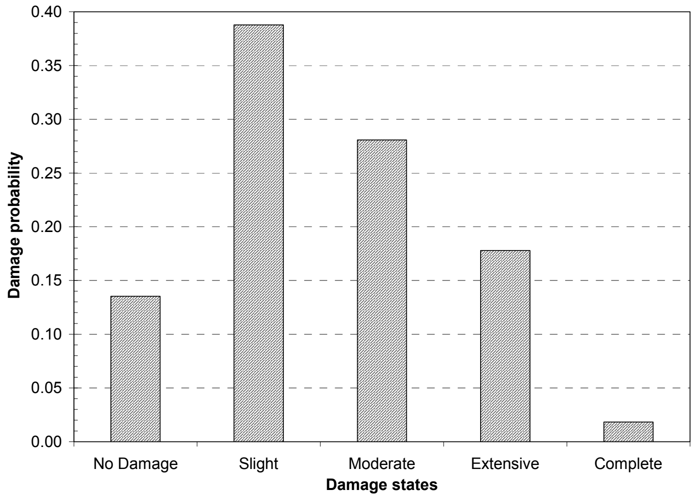

Figure 7. Histogram with the damage probability for each damage state.

6. Conclusions

An advanced method has been used in this paper to assess the vulnerability of a typical building of the Eixample district of Barcelona, Spain. The method is an update of the capacity spectrum method and proved to be adequate in evaluating the seismic behaviour of the bulding. The results show a high probability of ocurring moderate to severe damage of the studied building.

Acknowledgments

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NEW TECHNIQUES APPLIED TO POST-EARTHQUAKE ASSESSMENT OF BUILDING

Martha L. CARREÑO[1], Omar D. CARDONA[2] and Alex H. BARBAT[3]

Abstract

After a strong earthquake the damage in the affected area can be so extended that it is not possible to make all building evaluations only by expert engineers. It is common the tendency of non-expert inspectors to aggravate or to underestimate the real level of damage. But, due to the fact that the damage levels are usually linguistic qualifications such as light, minor, moderate, average, severe, etc., an expert system implemented in a computer for post-earthquake evaluation of building damage has been developed using an artificial neural network and fuzzy sets technique. This expert system allows performing the building damage evaluation by non-experts that participate in a massive survey of buildings. The model considers different possible damages in structural and architectural elements and potential site seismic effects in the ground. It takes also into account the pre-existing conditions that can make the building more vulnerable, such as the quality of construction materials, plant and height irregularities and bad structural configurations. The system makes decisions about the building habitability and reparability applying fuzzy rule bases to the available building information.

The global level of the building damage is estimated taking into account the structural and non-structural damage. The global building state is determined adding the rule base on ground conditions, obtaining thus the habitability of the building. The building reparability also depends on other fuzzy rule base: the pre-existent conditions. Thus, the expert system aids to make decisions on habitability and reparability of each building that are basic in the emergency response phase after the occurrence of a strong earthquake.

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1. Introduction

In case of a strong earthquake, the damage evaluation process must be made by a broad group of professionals related to building construction. It is highly desirable that people involved in this process have expertise and experience in these tasks. Nevertheless, the professionals having these skills are usually only a few and it is necessary to involve inexperienced voluntary engineers or architects. As a consequence, the damage underestimation or overestimation is common. Therefore, this work proposes the use of the computational intelligence as support to this task, developing an expert system for supporting the building damage evaluation process, using artificial neural networks and fuzzy sets.

2. Damage evaluation after an earthquake

As a result of earthquakes occurred in different countries located in seismic areas, the development of guidelines to damage evaluation in buildings has been necessary, with the aim of deciding as soon as possible whether the buildings may continue being used or not. After a strong earthquake, the identification of the constructions which suffered serious damage, and that can represent thus a danger for the community, is crucial. The identification of the safe constructions that can be used as temporary shelters for evacuated people is also necessary. Some countries have developed systematic guidelines and procedures to evaluate the building damage, namely Mexico[1, 2], Japan[3], United States[4], Italy[5], Macedonia[6] (former Yugoslavia), Colombia[7, 8, 9], among others. Damage evaluations are useful to improve the effective earthquake-resistant construction codes, by identifying the type of failure of the structural systems.

2.1. Problems with the damage evaluations

When the damage in the area struck by an earthquake is extensive, local experts in earthquake engineering are always insufficient to make all the evaluations on the state of the buildings. Professionals with little or no experience must be part of the evaluation teams. According to the findings of risk perception researchers, the tendency of inexpert inspectors is to aggravate or to underestimate the damage level. The information on the damage evaluation is highly subjective and depends on heuristic criteria and the biases of introduced by the inspectors in each case. The damage levels are defined in all evaluation guidelines using linguistic qualifications like light, moderate, severe or strong; these concepts may have different meanings according to the judgment of each person and a defined limit between these assessments does not exist clearly.

3. Computational modeling for post-earthquake damage evaluation

The problems that appear in the process of damage evaluation suggested to the authors to look for new tools that facilitate the work. The proposed model uses the fuzzy logic approach motivated by the incomplete and subjective character of the information. Post earthquake damage evaluations use qualitative and linguistic expressions that are appropriately handled by the fuzzy sets approach. On the other hand, an artificial neural network (ANN) is used to calibrate the expert system using the criterion of specialists. This enables the use of computational intelligence for the evaluation of damage by neophytes. For the model development, several building damage evaluation guidelines were taken into account. In addition, several members of the Colombian Association for Earthquake Engineering technically supported this work. The model has been implemented as a Visual BASIC 6.0 computer program, and has been called Earthquake Damage Evaluation of Buildings, EDE.

3.1. Artificial Neural Network structure

The ANN has three layers. The variables in the input layer of the neural network are grouped in four types, namely structural elements (SE), non-structural elements (NE), ground conditions (GC), and pre-existent conditions (PC). Each one contributes with information to neurons in the intermediate layer; they only affect the intermediate neurons in the group to which they correspond. The number of input neurons or variables in the model is not constant; it depends on the class of the structural system that will be evaluated and on the importance of the different groups of variables selected for the evaluation. The number of neurons of the input layer of the structural elements group changes according to the class of building. Table 1 presents the structural elements or variables considered according to the structural system. A qualification is assigned depending on the observed damage, using five possible damage levels that are fuzzy sets. For structural and non-structural elements, the following linguistics damage qualifications are used: none (N), light (L), moderate (M), heavy (H) and severe (S). Figure 1 illustrates the membership functions for these qualifications. The fuzzy sets are based on selected damage indices (section 3.2). Damage in the non-structural elements do not endanger the stability of building, but may represent a hazard for the occupants. The non-structural elements are classified in two groups: common and optional elements. Table 2 illustrates the groups.

Table 1: Structural elements according to the structural system

Structural System	Structural Elements
RC frames or (with) shear walls	Columns/walls, beams, joints and floors
Steel or wood frames	Columns, beams, connections and floors
Unreinforced/Reinforced/Confined masonry	Bearing walls and floors
"Bahareque" or "tapial" walls	Bearing walls and floors

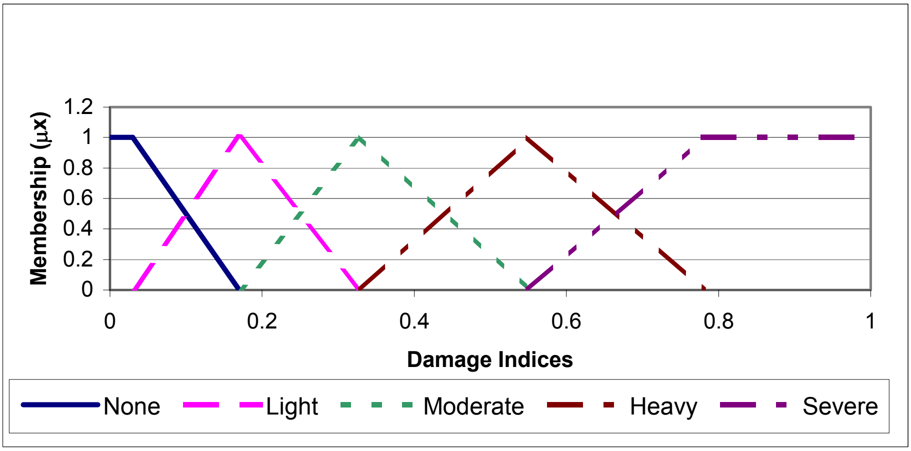

Figure 1: Membership functions for linguistic qualifications

Table 2: Non-structural elements

Common Elements	Partitions
	Elements of facade
	Stairs
Optional Elements	Ceiling and lights
	Installations
	Roof
	Elevated tanks

The variables of the ground and pre-existent conditions are valuated through the qualification of their state at the evaluation moment. The linguistic qualifications are: very good (VG), good (G), medium (M), bad (B), and very bad (VB). The ground conditions comprise the occurrence of landslides and soil liquefaction. Pre-existent conditions are related to the quality of the materials of construction, plane and vertical shape irregularities of building, and the structural configuration. In the intermediate layer, one index is obtained by defuzzification of each group of variables. Taking into account the four available indices, it is possible to define in the output layer the building damage using fuzzy rules with the structural and non-structural evaluations. The building habitability is obtained also involving the assessment of the ground conditions. Finally, using the pre-existent conditions it is possible to define the required level of reparability.

3.1.1. Input layer of the ANN

The fuzzy sets for each element or variable *i* (for instance columns or walls), in the input layer, are obtained from the inspector's linguistic qualifications of damage D_j

in each level j and its extension w_j. The damage extension (percentage of each damage level in each element) varies from 0 to 100 and it is normalized

$$w_j = \frac{D_j}{\sum_N D_j}, \sum_N w_j = 1 \tag{1}$$

The aggregated qualification of damage D_i for each variable is obtained with the union of the scaled fuzzy sets, taking into account the damage membership functions $\mu D_j(D_j)$ and its extensions or weights assigned by the inspector

$$D_i = (D_N \cup D_L \cup D_M \cup D_H \cup D_S) \tag{2}$$

$$\mu_{D_i}(D) = \max\left(w_{N,i} * \mu_{D_N}(D_{N,i}), \ldots, w_{S,i} * \mu_{D_S}(D_{S,i})\right) \tag{3}$$

Union in the theory of the fuzzy sets is represented by the maximum membership or dependency. By means of defuzzification, using the centroid of area method (*COA*), a qualification index C_i is obtained for each variable of each group of neurons

$$C_i = \left\lfloor \max\left(w_{N,i} * \mu_{D_N}(D_{N,i}), \ldots, w_{S,i} * \mu_{D_S}(D_{S,i})\right)\right\rfloor_{centroid} \tag{4}$$

3.1.2. Intermediate layer of ANN

In this layer, there are four neurons corresponding to every group of variables: structural elements, non-structural elements, ground conditions, and pre-existent conditions. Figure 2 shows a general scheme of the evaluation process. In this model of neural network, the inputs of the four neurons are the qualifications C_i obtained for each variable of the each group of neurons and its weight W_i, or degree of importance on the corresponding intermediate neuron introduced by the inspector according to its own criteria. These weights are normalized and are calibrated by means of a learning function (section 3.2). The initial values and the training process of theses weights have been defined and made by the participation of experts in earthquake damage evaluation. Using these qualifications and weights of each variable i, a global index could be obtained, for each group k, from the defuzzification of the union or maximum membership of the scaled fuzzy sets. The membership functions $\mu C_{ki}(C_{ki})$ and their weights W_{ki}

$$\mu_{CSE}(C) = \max\left(W_{SE\,1} * \mu_{C_{SE\,1}}(C_{SE\,1}), \ldots, W_{SEi} * \mu_{C_{SEi}}(C_{SEi})\right) \tag{5}$$

$$I_{SE} = \left\lfloor \max\left(W_{SE\,1} * \mu_{C_{SE\,1}}(C_{SE\,1}), \ldots, W_{SEi} * \mu_{C_{SEi}}(C_{SEi})\right)\right\rfloor_{centroid} \tag{6}$$

show the notation for the group of structural elements. The groups of variables related to ground and pre-existing conditions are optional then they can be or cannot be considered within the evaluation. If this happens, the habitability and reparability of building is assessed only with the structural and non-structural information.

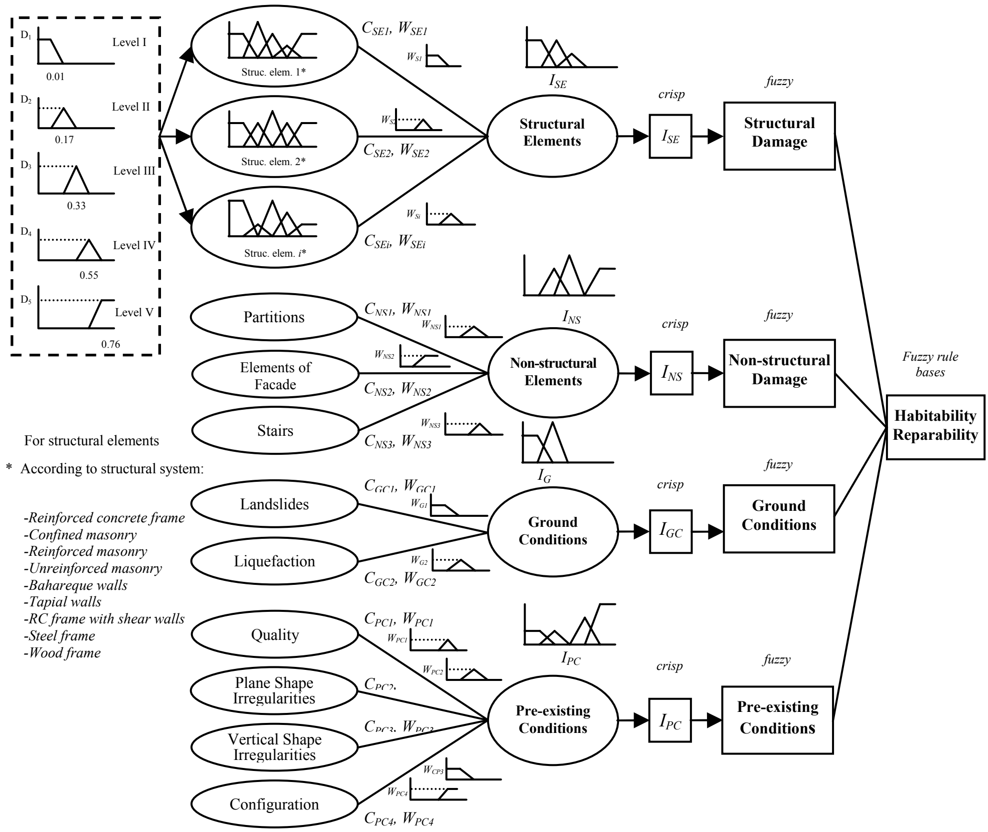

Figure 2: Structure of the proposed ANN

3.1.3. Output layer of the ANN

In this layer the global indices obtained for structural elements, non-structural elements, ground and pre-existent conditions correspond to one final linguistic qualification in each case. The damage level is obtained according to the "proximity" of the value obtained to a global damage function of reference. In this layer, it takes place also the process of training of the neural network. The indices that identify each qualitative level (centre of cluster) are changed in agreement to the indices calculated in each evaluation and with a learning rate. Once the final qualifications are made, it is possible to determine the global building damage, the habitability and reparability of the building using a set of fuzzy rules bases.

3.2. Learning process of the ANN

The output layer of the neuronal network is calibrated when the damage functions are defined in relation to the damage matrix indices. In order to start the calibration, a departure point is defined, that means the initial indices of each level of damage. The indices proposed by the ATC-13[10], Park, Ang and Wen[11], the fragility curves used by HAZUS-99[12], and the indices used by Sanchez-Silva[13] have been considered. The values of these indices correspond to the area of the centroides of each membership function related to each damage level. Table 3 shows the indices proposed in this work, which can be compared with those proposed by Park, Ang and Wen and Sanchez-Silva. The selection of the initial indices is based on those of Park; this choice can be justified on the basis that they have been calibrated with information of several studies. Those authors consider that collapse occurs in 0.8, although Stone[14] propose a collapse threshold of 0.77. Considering this, 0.76 is the selected index for the destruction level or collapse. In the selection of the damage index, the authors decided to be conservative, since the indices corresponding to severe and moderate damage have been highly discussed, and doubts exist on whether they should be smaller.

Table 3: Comparative table for damage indices

Damage Level	Park, Ang and Wen	Sanchez-Silva	Proposed
Very light	< 0.1 0.07	0.10	0.07
Light	0.10 – 0.25 0.175	0.20	0.17
Moderate	0.25 – 0.40 0.325	0.35	0.33
Severe	0.40 – 0.80 0.6	0.60	0.55
Destruction	>0.80 0.8	0.90	0.76

The calibration is performed for each damage level and only the indices corresponding to the groups of variables considered in each evaluation are calibrated. The network learning is made using a Kohonen network

$$I_{kj}(t+1) = I_{kj}(t) + \alpha(t)\left[I_{kj}(t) - I_{kj}\right] \tag{7}$$

where I_{kj} is the value of the index of a group of variables k recalculated considering a learning rate α and the difference between the resulting index of the present evaluation and the previous indices in each level of damage j. The learning rate is defined by

$$\alpha(t) = 0.1 * Exp(-0.1 * t) \tag{8}$$

where t is the number of times that has been used the index or weight that is calibrated. For training, the damage evaluations made during the Quindío's earthquake in Colombia (1999) were used. The neural network has been calibrated for reinforced concrete framed buildings, however more information is necessary to complete the network training for other structural classes, such as the wood and steel frame structures, because these building classes are not common in that area struck by the earthquake. Reinforced concrete frames with shear walls were only a few also, therefore the number of building evaluations to calibrate this structural system were insufficient.

3.3. Fuzzy rule bases

Once obtained the damage level of the structural and non-structural elements, the state of the ground and pre-existent conditions, the habitability and the reparability of the building are assessed. Figure 3 displays the fuzzy rule bases used. The global level of building damage is estimated with the structural and non-structural damage results. This has five possible qualifications: none, light, moderate, heavy and severe damage. The global building state is determined taking into account the rule base of ground conditions and thus the habitability of the building. The linguistic qualification for the building habitability has four possibilities: usable, restricted, prohibited and dangerous. They mean habitable immediately, usable after reparation, usable after structural reinforcement, and non-usable at all. Besides, the building reparability depends on another fuzzy rule base: the pre-existent conditions. The building reparability has also four possibilities: not any or minor treatment, reparation, reinforcement, and possible demolition.

4. Conclusions

- After a review of different guidelines for post-earthquake building damage evaluation, an innovative expert system has been proposed. The distinct advantages and disadvantages of each method were considered for the development of the tool.
- The expert system was developed by using artificial neural networks and fuzzy logic approach in order to improve the existing field methodologies. This type of tool is very appropriate in the practice, due to the subjective nature of the building damage evaluations and the incomplete information.
- The evaluations made by expert engineers after the earthquake of Quindío, Colombia, in 1999, have been very useful for the expert system training.
- The use of AI tools in Civil Engineering has very little diffusion until present, thus it is recommended to promote their use to provide suitable and versatile solutions to different problems in this field of knowledge.

5. Acknowledgements

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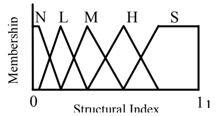
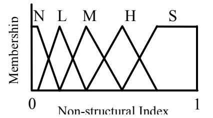


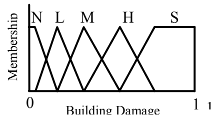
Structural Damage



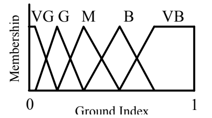
Non-structural Damage

Structural \ Non-structural	None	Light	Moderate	Heavy	Severe
None	N	L	L	M	M
Light	L	L	M	M	M
Moderate	M	M	M	H	H
Heavy	H	H	H	S	S
Severe	S	S	S	S	S

Rule base for Building Damage
Structural Damage *vs.* Non-structural Damage



Building Damage



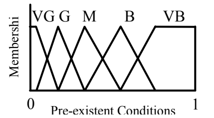
Ground Conditions

Building Damage \ Ground Conditions	Very Good	Good	Medium	Bad	Very Bad
None	N	L	M	H	S
Light	L	L	M	H	S
Moderate	M	M	H	S	S
Heavy	H	H	S	S	S
Severe	S	S	S	S	S

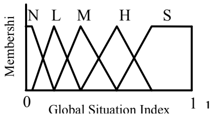
Rule base for Building Situation
Building Damage *vs.* Ground Conditions

Building Damage \ Ground Conditions	Very Good	Good	Medium	Bad	Very Bad
None	U	U	R	P	D
Light	U	U	R	P	D
Moderate	R	R	P	D	D
Heavy	D	D	D	D	D
Severe	D	D	D	D	D

Rule base for Building Habitability
Building Damage vs. Ground Conditions



Pre-existent Conditions



Building Situation

Building Situation \ Pre-existent Conditions	Very Good	Good	Medium	Bad	Very Bad
None	M	M	M	R	S
Light	M	M	R	S	S
Moderate	R	R	S	S	S
Heavy	S	S	S	D	D
Severe	D	D	D	D	D

Rule base for Building Reparability
Building Situation vs. Pre-existent Conditions

NOTATION:

Structural and Non-structural damage:
N: None
L: Light
M: Moderate
H: Heavy
S: Severe

Ground and Pre-existent conditions:
VG: Very good
G. Good
M: Medium
B: Bad

Habitability:
U: Usable
R: Restricted
P: Prohibited
D: Dangerous

Reparability:
M: Not any or minor
R: Reparation
S: Strengthening
D: Possible

Figure 3: Method for Building Habitability and Reparability.

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Iasi, Romania, October 24, 2003

SEISMIC FRAGILITY CURVES FOR FRAMED BUILDINGS WITH FLAT BEAMS

Rosangel MORENO[1], **Alex BARBAT**[2] **and Lluis PUJADES**[3]

Abstract

Procedures and techniques for the study of the seismic behaviour of buildings base on existing computer programs are developed in this paper. SAP2000 non-linear computer program was used to design the building. STAC computer program was applied to consider the uncertainties in the mechanical properties of the materials and to perform Monte Carlo simulations; this program enables generating data samples for geometrical and the material properties of a structure considered as random variables. RUAUMOKO computer program was employed to assess the seismic behaviour of the structure by means of a non-linear static analysis, calculating its capacity spectra. BCSEC computer program calculates the interaction and the moment-curvature diagrams for each cross section of the structural elements required as an input data by RUAUMOKO program.

A stochastic analysis of the capacity and seismic response was carried out and fragility curves were then obtained for a five stories framed building using a lognormal cumulative function, indicating the probability of reaching or exceeding a certain damage state, for a given seismic demand. The main application of these curves is found in the simulation of vulnerability and seismic risk scenarios for urban areas.

1. Introduction

The evaluation of the seismic vulnerability is very important to predict the seismic damage in existing buildings. Despite the large number of researches performed in this field, there are still many questions to be answered. A methodology for assessing the seismic behaviour of framed structures is developed in this work, its main results consisting of fragility curves.

The RC buildings subjected a seismic load have uncertainties in various parameters which characterize their behaviour, reason for which a stochastic analysis has been carried out. The seismic behaviour of a five stories RC framed building with flat beams has been analysed, being this a construction type which is common in many countries threatened by earthquake.

To evaluate the seismic capacity of the structures, a non-linear static analysis was performed, the capacity curve and thus the capacity spectrum being obtained. It

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was possible to compare these curves with the demand spectra; the intersection of these two spectra, which defines the maximum response of the structure, is called “performance point”. The seismic demand spectrum includes the effects of energy dissipation due to the inelastic behaviour of the buildings when subjected to strong earthquakes, and the current degradation of the structure. Finally, fragility curves are developed which permit obtaining the seismic vulnerability of the building. The main objective in this work is to develop this methodology based on the mentioned computer programs and to assess the vulnerability of RC buildings designed with Spanish codes for low to moderate seismicity zones, like the case of Barcelona, Spain.

2. Structural typology and structural analysis

The typology studied herein is a five stories framed reinforced concrete building with flat beams. The building has a rectangular plan with 17.00m×12.00m and 15.80m of maximum height. The cross sections of the columns have 30cm×30cm or 35cm×35cm, those of the beams have 25cm×25cm while the slab has a height of 25cm (Figure 1). The mechanical properties are: strength of the concrete in compression f′c=25 N/mm^2 and the steel yield stress fy=400 N/mm^2. The program SAP2000 was used to design the building taking into account the Spanish building codes and the service and ultimate limit states specified in these codes.

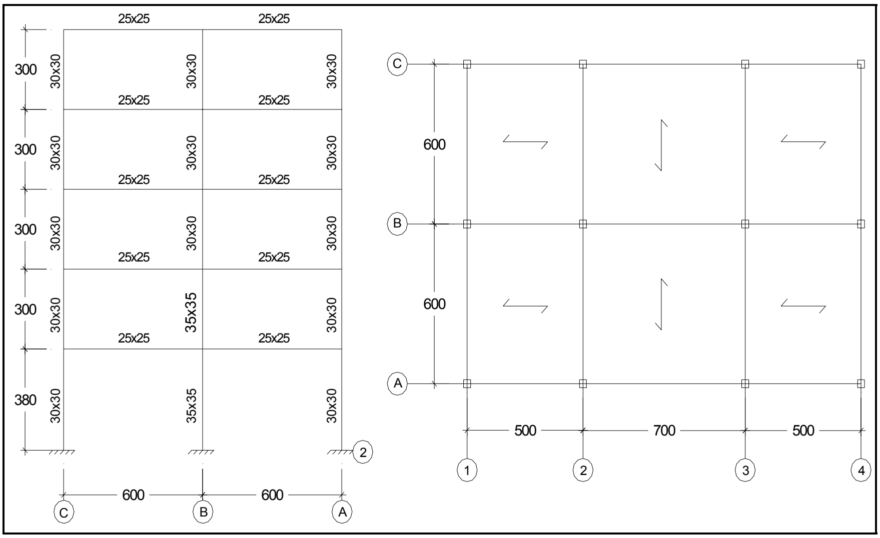

Figure 1. Typical frame and rectangular plan (cm).

The wind action has been modelled as quasi-static while the earthquake action has been modelled by using the modal superposition analysis, for a 3D model with 15 degrees of freedom (3 by story).

3. Non-linear analysis

A complete evaluation of the stiffness, strength and ductility of the structure was performed by means of a non-linear analysis, by using RUAUMOKO (Carr, 2000), BCSEC (Bairán, 2000) and STAC (*"Stochastic Analysis Computation"*, 2002) programs. RUAUMOKO is a non-linear static and dynamic program in 2D which requires modelling the structure by means of several plane frames, connected each to the other (Figure 2). To introduce the effect of the rigid diaphragm, the nodes of the same storey were constrained. BCSEC is a computational program to calculate the non-linear mechanical characteristics of cross sections. In this program the different materials composing the section are defined, the corresponding stress-strain curves are assigned; in this case parabolic-rectangular and linear-rectangular curves have been assigned to the concrete and steel, respectively (Figure 3). STAC is a stochastic analysis computation program, which performs Monte Carlo simulations. The distribution functions for the mechanical characteristics of the materials were defined and variation coefficients were assigned (Yépez, 1996). The distribution type, variables and the values of the parameters that characterize each coefficient are shown in Table 1 (Moreno et al. 2003a).

Table 1. Random variables.

Variables	Distribution	Mean	COV
fc_k	Normal	25 N/mm^2	0.15
Ec	Normal	3.21e^4 N/mm^2	0.15
fy_k	Lognormal	400 N/mm^2	0.11
Es_{max}	Normal	2.1e^5 N/mm^2	0.09
fy reinforcement	Lognormal	400 N/mm^2	0.11

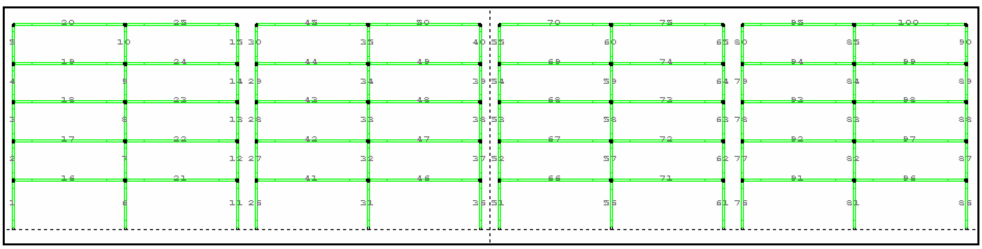

Figure 2. Frames for the analysis in RUAUMOKO-2D.

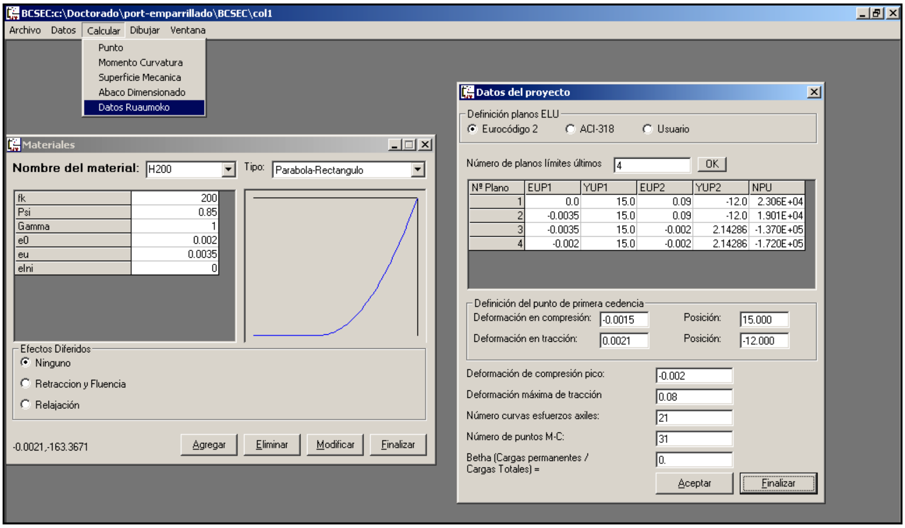

Figure 3. Definition of the materials and other project data.

4. Structural capacity and seismic demand

The structural capacity is calculated by means of a non-linear static analysis with the RUAUMOKO program, by applying a monotonic increasing load to the structure until the ultimate load is reached. The loading pattern chosen in this work corresponds to the seismic loading for the equivalent static analysis given by the Spanish code (NCSE-94). Thus a pushover curve that relates the base shear (V) to the roof displacement (Δ_{roof}) is obtained, as it can be seen Figure 4. The pushover or capacity curve is transformed by using spectral coordinates, according to equations (1) and (2), obtaining thus the capacity spectra of Figure 5

$$Sa = \frac{V/W}{\alpha_1} \tag{1}$$

$$Sd = \frac{\Delta_{Tope}}{PF_1 * \Phi_{1,Tope}} \tag{2}$$

In these equations, *Sa* and *Sd* are acceleration and displacement spectra, respectively, *W* is the modal weight, $\Phi_{1,Tope}$ is the amplitude of mode 1 at roof level and, α_1 and PF_1 are, respectively, the modal mass coefficient and modal participation factor for the first natural mode.

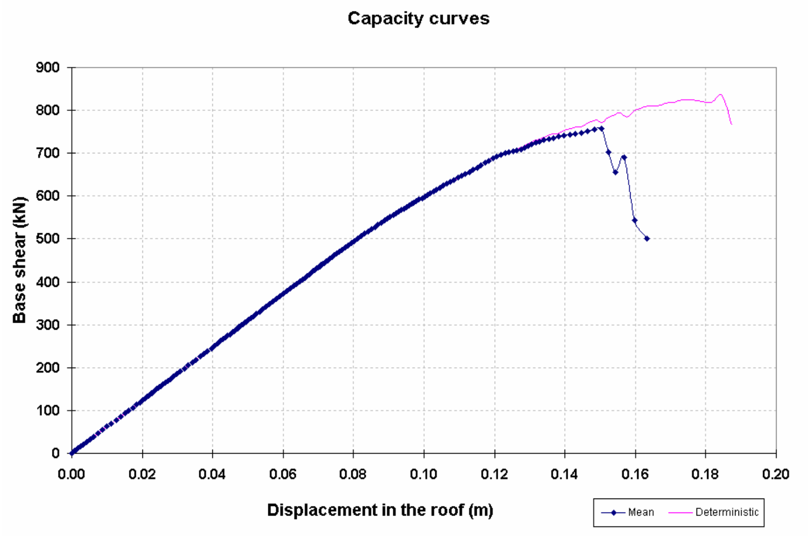

Figure 4. Capacity curves.

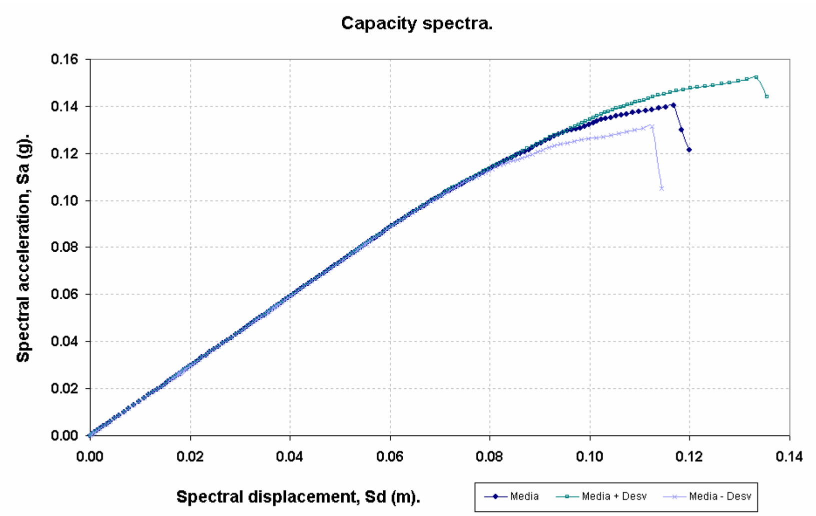

Figure 5. Capacity spectra.

The demand spectra were obtained by means of the Capacity Spectra Method (ATC-40, 1996), starting from the elastic spectra contained in the Spanish seismic code and the capacity spectra previously established. The maximum structural displacement expected for the seismic demand is the intersection of both spectra and was obtained by means of an iterative process (Moreno et al. 2003b). In Figure

6, the performance point, the demand spectra and the bilinear representation of the capacity spectrum for different PGA's are shown.

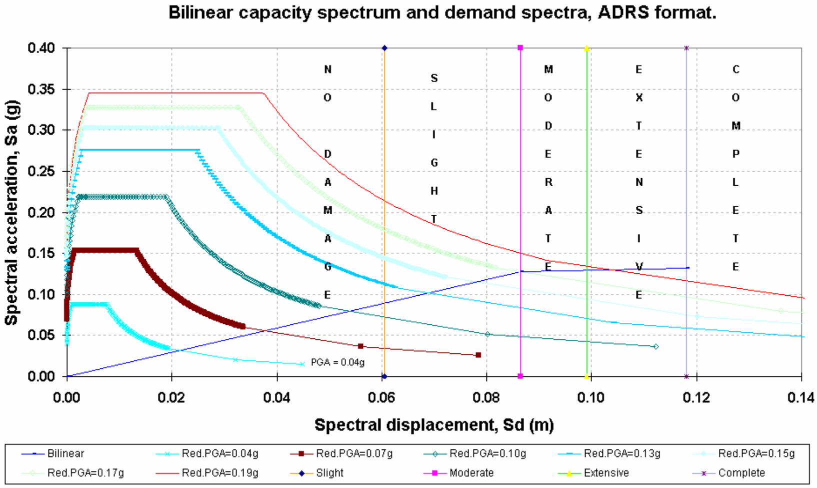

Figure 6. Bi-linear capacity spectrum and family demand spectra.

5. Fragility curves

The fragility curves can be defined by the mean value of the spectral displacement ($\overline{Sd}_{DS}$) corresponding to the threshold damage state (DS) and by the variability (β) associated to the same threshold of DS. The fragility curves are obtained in function of β_{DS} and $\overline{Sd}_{DS}$ using a lognormal cumulative function. The fragility curves give the probability of exceeding or reaching a certain DS: slight, moderate, extensive and complete, for a demand parameter which, in this case, is the spectral displacement (Sd) corresponding to the peak ground acceleration (PGA) (see equation 3 and Figure 7)

$$P\left[DS \geq DS_i / PGA\right] = \Phi\left[\frac{1}{\beta_{DS}} \cdot \ln\left(\frac{Sd}{\overline{Sd}_{DS}}\right)\right] \tag{3}$$

where (Φ) is the standard cumulative distribution function.

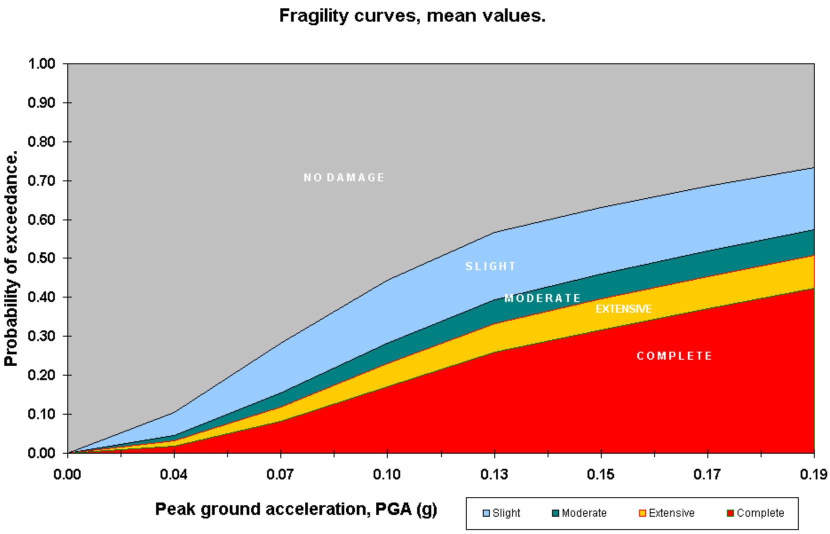

Figure 7. Fragility curves.

6. Conclusions

The methodology developed herein has been used to develop seismic damage scenarios for Barcelona, Spain, by means of fragility curves. The probability of damage of the structures was estimated by using Monte Carlo simulations.

The results shown in the paper indicate that the response of the building lies within the elastic range for accelerations smaller than a 10% of the acceleration of the gravity, while for accelerations near to 20% of g the building could suffer strong damage or even collapse. The building has a probability of about 40% to suffer collapse due to its low ductility.

Acknowledgments

This work has been partially sponsored by the Spanish Ministry of Science and Technology and with FEDER funds (projects: 2000-1740-C05-01/RIES, REN 2001-2418-C04-01 and REN2002-03365/RIES) and by the European Commission through the 5th Framework Programme Environment and Sustainable Development (RISK-UE Project, contract EVK4-CT-2000-00014).

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Iasi, Romania, October 24, 2003

THE AUTOMATIZATION OF THE STRUCTURAL CALCULATIONS

Nicolae MALCOCI[1], Ion COCIORVA[2]

Abstract

This paper presents the test of the bridge, obtained through theoretical calculation, test and simulation on computer.

Comparation of results shows that in the future is likely to replace test of the sridge and utiliye simulation on computer.

1. Introduction

At present, the domain of art works (bridges) is developed more than ever. The complete realization of a bridge is equal with a sequential process that implies the solving of multiple aspects such as economical, technical, social and environmental. A modality of reducing the costs of a bridge (its building and maintaining) represents its rational projection, based on exact calculations, including a mechanical model equivalent with its real structure. The best results are obtained in the cases when the calculations are based on models of numeric calculation. At present, the most frequently used method for the determination of a bridge's effort and deformation's state in the period of functioning is that of finite elements, with direct applicability by the help of the automatic calculation programs. According to this method, the structure is approximated by means of a finite system of one-dimensional, bi-dimensional and tri-dimensional discrete elements. By modeling, the degree of approximation of real structure under loading can be improved by increasing the number of component elements, concomitantly with diminishing their dimensions. The elements' standard form leads to type-expressions of different mechanical measures, this way, giving the opportunity to automatize the calculations.

2. Several considerations in determining the safety and resistance of highway bridges

The lifetime duration of a building is a relative notion and can be defined as the interval of time during which the structure satisfies the conditions of functioning, without exaggerate expenses for maintaining it according to the parameters necessary for an extremely safe exploration.

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The safety of a construction is defined as the probability of a safe functioning and reaction to future actions from exterior on the whole period of its duration, on condition that the construction meets the requirements imposed through projection, taking into account the resistance and exploitation.

The following factors contribute to it: the selected constructive system, the utilized materials, the technology of planning and the modality of application, mechanical and environmental actions, the modality of maintaining and reparation etc. All of these contribute, in various proportions to the sustainability of the building and the maintaining of its aptitude for a normal exploitation.

According to the standards and the instructions existing in the Republic of Moldova the technical situation of bridges is determined in the result of a detailed expertise (once in 5 years). In case that the examined building presents drawbacks that affect its resistance, they decide making tests of the bridge.

At present, testing the bridge is a process requiring time and considerable financial means. That is why, in the present paper, we plan to analyze the possibility of substituting the classical attempt (test) with a simulation of a bridge's functioning. By simulation we mean a technique of realizing the experiments on computer, which fact implies building a series of mathematical and logical models, by means of which is described the functioning of a real system during a longer period of time. The necessity of simulation is deduced from the fact that, in the most of the cases, real systems cannot be studied directly because of their complexity.

For simulation, they selected the bridge on the national road R 28 Rascaietii-Noi – Hlinaia traversing the river Nistru. The respective bridge is a construction of 5 simply leaned openings (2x33+2x42+33) (picture 1) and elastic infrastructures in form of frame and two pylons. The bridge's total length is of 185 meters, the gauge of 10,15 meters and 2x1,5 m. pavements.

Picture 1. General view of the bridge from the right bank of the river

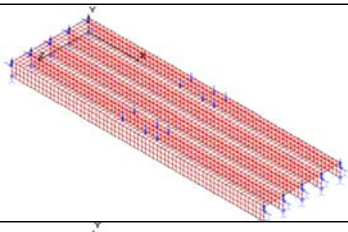
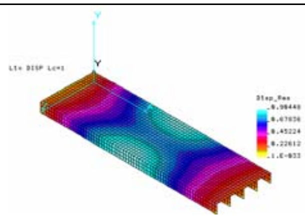
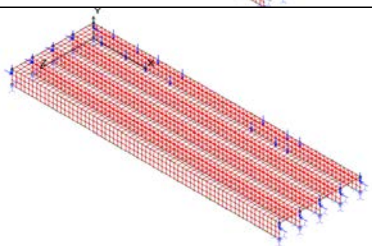
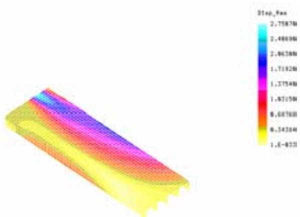
Due to the fact that after approximately 20 years of bridge's exploitation they ascertained numerous degradations, in September 2002, they made the bridge's expertise according to the technical standards СНиП 3.06.07-8, and its testing (the openings 4 and 5) in 9 loading hypotheses (picture 2).

Picture 2 Loading the bridge with the test convoy

Below, we present the results of the simulation of the loading hypotheses Nr. 5-9 on the 5th opening, which is made in transversal profile of 5 girders made of pre-compressed 33 meters length concrete. The bridge’s table was modeled from bi-dimensional elements with 4 knots on each element. It was modeled so that the knots coincided of the place of exterior forces action (the vehicles’ wheels).

The results obtained through simulation for the hypotheses 5 and 8 of loading are graphically represented in chart 1.

Chart 1 Graphical representation of results

The number of the loading hypothesis	Loading	Removals
5		
8		

In Chart 2 they present the values of removals in control points, results obtained by theoretical calculation, by measuring after loading with convoys (autocades) and by simulation.

Chart 2 Analytical representation of results

Loaded part	Loading hypothesis	Removal		
		measured	calculated	simulation
Right	5	0,98	1,2	0,904
Left		1,11	1,2	0,904
Right	8	1,82	1,85	1,75
Left		0,15	-	0,2

3. Conclusions

After effecting the simulation of two hypotheses of loading and comparing the obtained results with the measured results and the ones theoretically calculated we drew the following conclusions:

The utilized program of calculation is an efficient one, giving the possibility to model a series of complex structures;

- An exact calculation, with good results is obtained when using the computer. The user should have deep knowledge of the phenomena that are produced and develop in the analyzed structure and strong abilities in operating the calculation programs;
- The theoretical analysis of the structures is accomplished by the help of automatic calculation, based on complex programs, and the obtained results ought to be validated by significant tests, conclusive in number and contents;
- For achieving the economical, technical and esthetic performances of the bridges' structures, the projection must be effected according to a series of exact analyses and syntheses, in concordance with the engineering standards;
- The theoretical analysis of the bridges' structures with the help of computer and efficient programs reduce the time of investigations and, in some cases, the **References** obligation of effecting the respective tests on physical models, this sort of testing being extremely expensive.

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ANALYSIS OF DYNAMIC BEHAVIOUR UNDER TRAFFIC LOADS OF A STRENGHTENED OLD STEEL BRIDGE

Bogdan BOLDUS[1], Dorel BOLDUS[2], Radu BĂNCILĂ[3]

Abstract

Pe teritoriul de azi al Banatului, dar si in restul Romaniei, se gasesc inca un insemnat numar de poduri metalice, atat de cale ferata cat si de sosea, construite in perioada de la sfarsitul secolului al XIX^{lea} si primele decade ale secolulyui al XX^{lea} cand tinutul se afla sub administratie austro-maghiara. Desi dupa reglementarile actuale durata normata de exploatare a unui astfel de pod este limitata la 100 de ani, multe dintre acestea nu au putut fi inlocuite la timp cu structuri noi, in principal din motive financiare, si ca urmare se afla inca in exploatare. Avand in vedere vechimea acestor structuri cat si conditiile actuale de exploatare, mult mai severe decat cele avute in vedere la proiectarea si executia lor, a devenit imperios necesara o verificare amanuntita a capacitatii lor portante pentru cunoastere si luarea masurilor necesare pentru mentinerea lor in exploatare in conditii acceptabile de siguranta. O situatie aparte o reprezinta podul de sosea peste Mures de la Savarsin. Acest pod situat pe drumul judetean DJ 707A, la km. 1+271, a fost construit in anul 1897 iar in anul 1997, in urma unei expertize tehnice neconcludente, a fost inchis circulatiei publice, in prezent fiind permisa doar circulatia pietonala si a autoturismelor usoare, structura metalica nebeneficiind de intretinerea curenta regulamentara. Aceasta stare de fapt creeaza serioase dificultati traficului rutier in zona dar mai ales activitatii economice din agricultura deoarece majoritate terenului agricol al comunei Savarsin se gaseste situat pe malul opus al Muresului iar cel mai apropiat punct de trecere al raului se gaseste la o distanta de 40 km. Imposibilitatea finantarii reconstructiei unui pod nou a condus la ideea reabilitarii suprastructurii podului existent pentru cerinte minime. Un astfel de demers a fost efectuat de colectivul de poduri metalice de la Universitatea "Politehnica" din Timisoara si s-a finalizat in anul 2000 cu un proiect tehnic de consolidare a podului cu urmatoarele solutii: inlocuirea caii actuale pe profile Zorés cu o placa din beton armat sustinuta de tabla cutata si in conlucrare cu grinzile caii, inlocuirea lonjeronilor marginali si consolidare directa a celor intermediari, consolidarea directa si indirecta a antretoazelor si a grinzilor principale, inlocuirea contravantuirii superioare deteriorate. In colaborare cu catedra de Mecanica constructiilor de la TU Muenchen, s-a efectuat si o analiza cu MEF, utilizand programul Nastran-MSC, a comportarii dinamice a podului reabilitat si simularea trecerii unui camion cu o anumita viteza. Principalele aspecte si concluziile rezultate sunt prezentate in lucrarea de fata.

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1. Introduction

The bridge over Mures river situated on the county road DJ 707A at Km. 1+271, near Savarsin, was built in 1897 as a steel structure with four span of 39,8 m. length. All spans were designed in a constructive art typical from historical period of XX^{-th} century begin: the main truss girder with parabolic upper chord and downward cross stud, longitudinal and transverse floor beams (stringers and crosssgirders) at lower chord, a ZORÉS profiled deck covered by road's system and lower and partially upper windbracings. In figure 1 is presented the general view of bridge and in figure 2 some pictures of old structure.

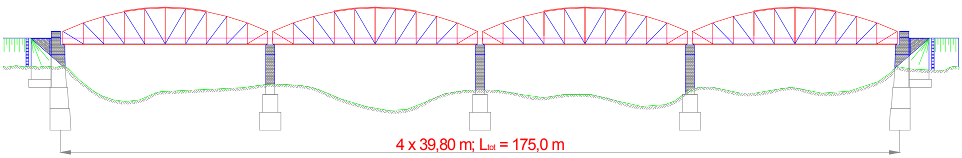


Figure 1. General view of Savarsin bridge

Figure 2. The old bridge structure

2. Bridge structure's rehabilitation

Due to the bridge's geometrical dimensions which don't satisfy the present conditions of side clearance it has arranged with road user to assure a single 4,0 m. width running way, which let run also the tractor-allied equipments and farm implements, and two 1,0 m. width pedestrian way near each main girder, separated from running way by a kerb and security sliding carriages. One-way traffic is regulated by street-traffic lights located at bridge's endings. After a detailed inspection and a design analysis of existing steel structure corroborated with prescriptions of design standards it was adopted different strengthening measures. Our rehabilitation strategy matches the adequate solutions of each structural element [1].

Bridge's deck rehabilitation. The bad status of roadbed and the strong corrosion of profiled steel deck as well as the great dead weight of bridge covering has contributed to decision of replacement of present deck by a new C20/25 reinforced concrete slab connected with welded head stud from the fresh added flange at upper side of stringers and crossbeams. The old classic way is going to be converted into a composite structure (fig.3 and fig.4) with a relevant elevated bearing capacity and an improved stiffness. In the aim to avoid the weld of connectors on an old reveted steel element, a new weldable flange was attached to stringers and crossbeams using high strength bolts. In the same time the crossgirder has been supplementary strengthened by a HEA tie member posted 200 mm. under crossbeam section. A slim profiled steel sheet was used as both formwork and supplementary reinforcement for concrete slab.

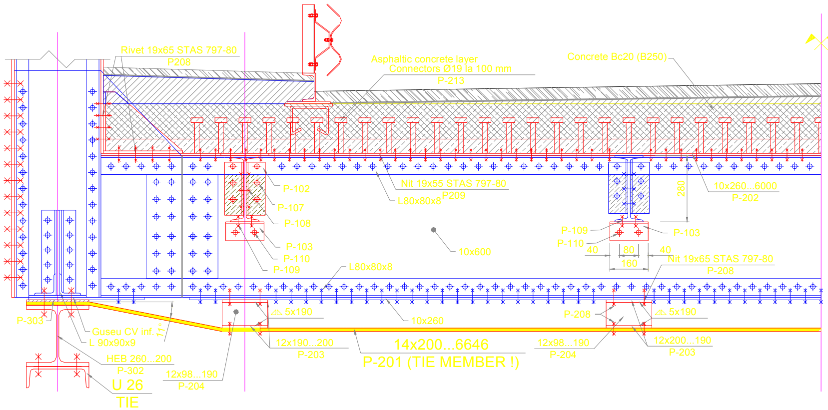


Figure 3. Floor beams cross-section

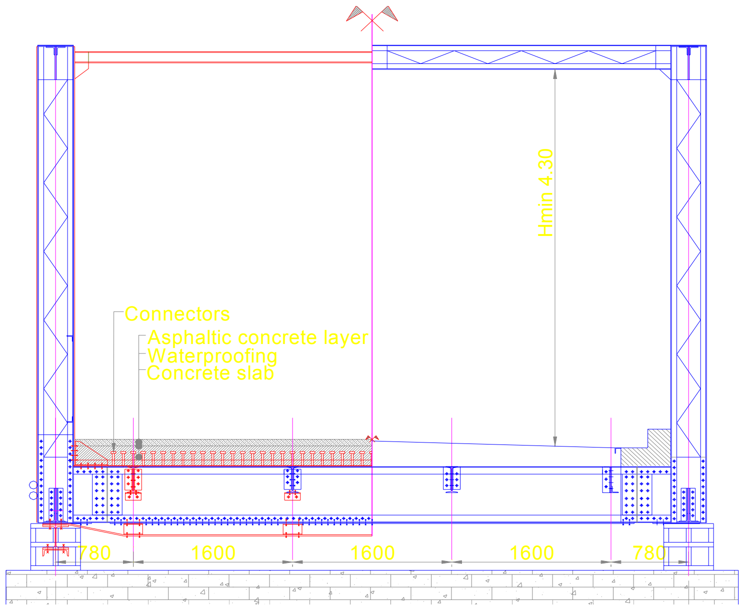


Figure 4. Bridge cross section

Main girders rehabilitation. For main girders a mixed reinforcing method was applied. That means: a) the direct strengthening of single wall cross-section of upper chord by adding two angle iron so it increase the member's second moment of area and reduce the element slenderness, specially from minor cross-section axis (fig.5a) and the direct strengthening of posts (form brace) by adding new flanges (fig.5b); b) the indirect strengthening by a rigid tie member posted under the bottom chord (fig.4); c) the replacement with new elements of upper windbracing which has been strongly crooked and damaged by over roadway clearance carriages (e.g. trucks loaded with big saw logs

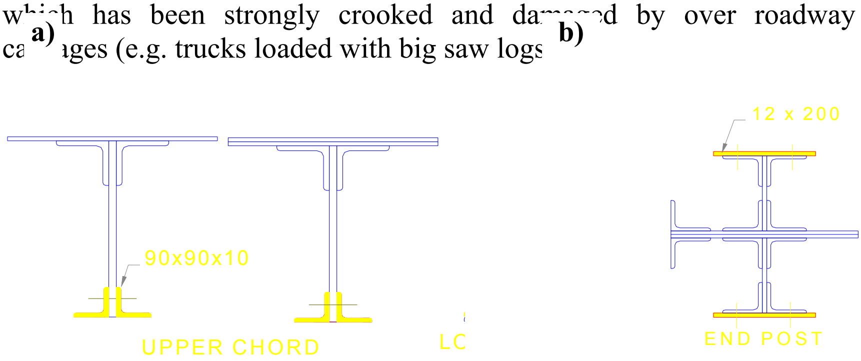


Figure 5. Reinforcement of main girder members: a) upper chord; b) posts

With this amendments the reinforced structure accomplish the thougness, stability and structural rigidity criteria given in design codes and service regulations.

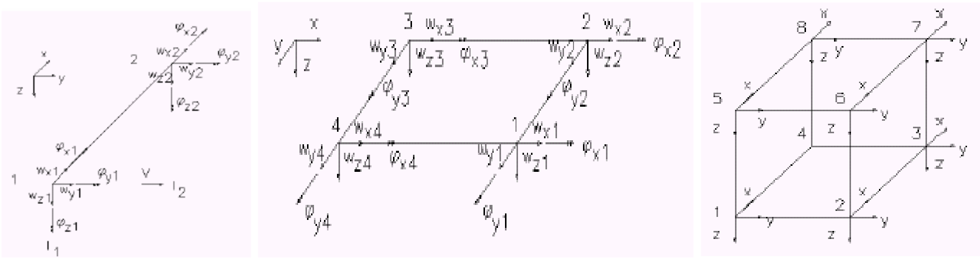
3. Dynamic analysis of strengthened structure

Inside a co-operation with TU München-Lehrstul für Baumechanik, the proposed strengthened structure has been analysed concerning the dynamic behaviour [2] under traffic loads. The analysis was accomplished with FEM, utilising MSC-NASTRAN program and the simulation with PRESIM 98, an oriented computer program created at TUM, in the aim to simulate the effect of a truck's going over the bridge. Due to the variety of structure's elements it was utilised in structure modelling different finite elements, such:

a) For the main girder's member, ties member, windbracing, crossbeam and stringer, the **BEAM-elements** are the dominating elements. This knows normal, torsion and bending general loads transfer (fig.6a.). The Beam-3d element has 6- degrees of freedom in each end and possesses an element matrix of 12×12.

b) The floor slab was simulated by **PLATE elements** (fig.6b). This element is a combined even shell element with a fixed thickness, which does not have to be however over the whole element, but is freely selectable at the corners. It knows normal, shearing and bending general load transfer. This element can be modelled also as triangle. The square PLATE element is a very complex element, there it 20 knot degrees of freedom and thus a 20x20-Elementmatrix possesses.

c) The shear connection designed as headed studs used with profiled steel sheeting were simulated by **SOLID-elements** (fig.6c). This element knows normal, bending and thrust general loads transfer. It has 8 knots and possesses as knot free values only shifts. Therefore moment loads can be applied only over auxiliary solutions. The element has only 4 degrees of freedom more and thus a 24x24-Elementmatrix.



a) b) c)

Fig.6. Charachteristic finite elements;
a) Beam element; b) Plate element; c) Solid element.

It was used in bridge modelling about 900 knots and 1286 finite elements. The entire modelled structure is shown in fig.7.

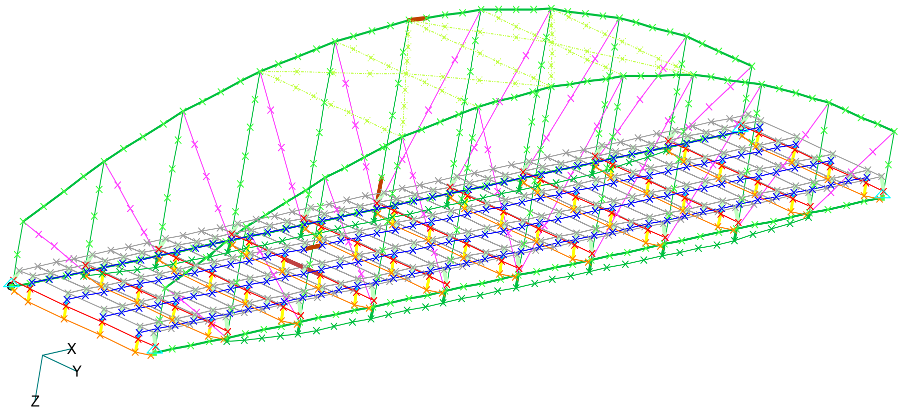


Figure 7. The entire modell of reinforced bridge structure

After CAD analysis of structure's model the first ten eigenmodes and his free-running frequencies have resulted. The normal frequency is 4.2 Hz and this show a stiffened bridge structure (fig.8).

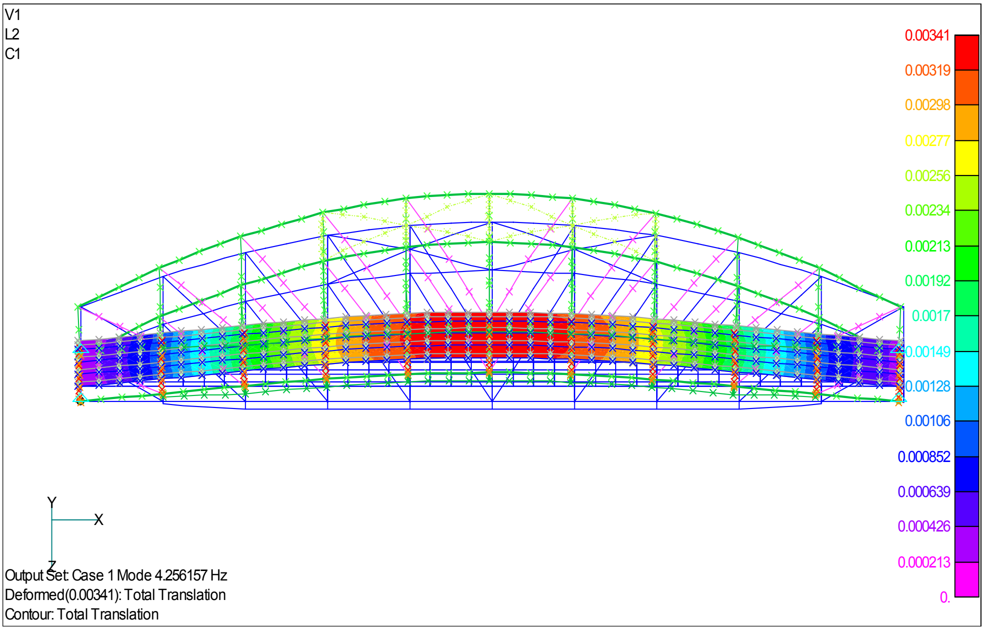


Figure 8. The first eigenmode and coresponding frequence

4. The truck's passage simulation

For the computation of passages of the different vehicle models out over the bridge model a pre-processor with the designation **"PRESIM 98"**, developed at TU Munich, has been used. In the context of their theses Fritsch (1994) [4], Lichte (1996), Neun (1998) and others [3],[5] already argued with the interaction and developed a pre-processor, that the conversion of the algorithm on Nastran input code automated. Bridge construction work and the vehicle is mechanically representable by its rigidity and mass matrix. If both models become directly in NASTRAN (and/or FEMAP) produced, then the stencils are built up automatically and the only problem for the users remains to model that material building and/or material vehicle suitably.

The wellknown linear equation from the building dynamics reads:

$$[\,M\,]\,\{\Phi^{''}\} + [\,C\,]\,\{\Phi^{'}\} + [\,K\,]\{\Phi\} = \{P\} \qquad (1)$$

whereby M the mass matrix, K the rigidity matrix, Φ the vector of the knot degrees of freedom and P the load vector. NASTRAN uses the symbol "s" as a time derivative operator, whereby (1) are also written as:

$$[\,Ms^2 + Cs + K\,]\,\{\Phi\} = \{P\} \qquad (1.a)$$

Interaction simulation with PRESIM 98, FEMAP and NASTRAN.

Vehicle and bridge should be produced first in two separated files. PRESIM 98 can process spatial vehicle models with maximally 8 axles. We have regarded an **A30** vehicle model after Romanian standards. For the input in PRESIM 98 the knot numbers of the left and right wheel trace as well as their distance from bridge beginning are to be noted to. Also it is necessary as input data the running way roughness. For the Savarsin bridge have no measured roughness, thus counted the interaction simulation on an ideal carriageway slab and the result will represent the corresponding influence line. In order to together-drive vehicle and bridge model in a file, bridge model (e.g. bruecke.mod) must be loaded first into FEMAP and then the vehicle model (e.g. truck.dat) to be read in. This vehicle-bridge system should be stored under its own file name (e.g. sys.dat).

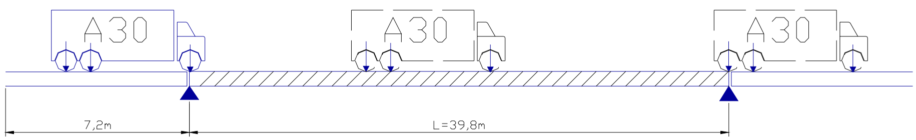
Static and dynamic passage simulation. The toes of the vehicle moves per time step a certain amount on the bridge-broad. This shift is however under normal conditions smaller than the modelled knot distances of the bridge. This has the consequence that the load is applied on the 2 edge knots of this element, because it is not possible an element between the knots to load. With it, like one is crucial this distribution of load assumes.

In the dynamic effect simulation two representative truck's running speeds were considered: a reduced one of 1 m/s (about 3,6 km/h) which represent the static loading and a higher one, of 14 m/s (about 50 km/h) which represent the speed of normal traffic over bridge as dynamic loading so it is shown in figure 9.

i) static passage v = 1 m/s (3,6 km/h)

initial loading	intermediately loading	end loading
t = 0 seconds.	ti = (0,1.... 36,9) seconds.	t = **47** seconds.

Computed intermediate load conditions $\Delta L = 0.1$m and $\Delta t = t_{i+1} - t_i = 0{,}1$ seconds.



ii) dynamic passage v = 14 m/s (50 km/h)

initial loading	intermediately loading	end loading
t = 0 seconds.	ti = (0,1.... 3,25) seconds.	t = **3,35** seconds.

Computed intermediate load conditions $\Delta L = 0.1$m and $\Delta t = t_{i+1} - t_i = 0{,}00712$ sec.

Figure. 9. Run travel of the A30-truck over the bridge

5. Conclusions

After input data computing has resulted, step by step, the knot's deplacement and stresses in structure elements and after Excel full-automatic processing, it has obtained the diagrams, like in figure 10, which shown, on time dependence, the dynamic effect.

Since both the vehicle and the roadway were idealistically regarded and, no differences were observed.

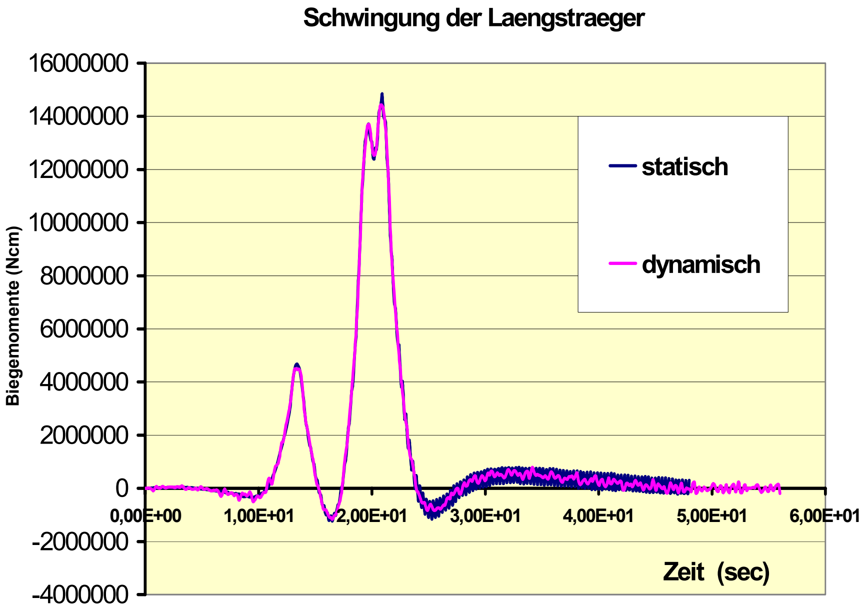


Figure 10. The dynamic effect of stringer

Additional analyses are recommended for the future with considering also:

- roadway imperfections those experimentally measured knew;
- truck natural oscillations.

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ANALYSIS OF STRESS STATE ON PLATES OF CONCRETE BRIDGES WITH PREFABRICATED BEAMS

Corina CHIOTAN[1]

Abstract

The calculation methods necessary for dimensioning the concrete bridges, in force in Romania, stipulate a simplified calculus for these ones by taking into account the ratio between plate sides. Thus, the plates supported on their contour, for which the sides ratio is bigger than 2, called long plates or plate bands, the calculation is made considering that significant bending moments could appear only on the direction of the plate short side. The calculus is made for plates with unitary width assimilated to simply supported beams on the short direction. The maximal bending moment, resulting according to this static scheme, is distributed between the middle of the plate and its support, through coefficients, which take into account the ratio between the beam height and the plate width. The fixing of plate on beams is not a perfect one, but an elastic fixing and the fixing degree of plates on beams is not quantified because the utilized coefficients are considered to cover all possible situations that could occur.

A detailed analysis of stresses acting upon reinforced concrete bridge plates could be achieved either through at site measurements, or using numerical methods of calculation. In order to validate some generally applicable results a great number of measurements on reinforced bridge plates are necessary, which means a special financial effort. Providing that the finite elements are correctly chosen, the analysis of bridge structures by means of the finite element method offers results much close to reality and the number of structures to be studied is practically unlimited.

The present paper presents the structural analysis of a reinforced concrete bridge with simply supported prefabricated beams made of prestressed concrete and the superstructure plates represent the interest area. The purpose has been to emphasize the maximal stress states in the bridge plates (bending moments on their short direction), dimensioning their characteristic sections, as well as the eventual appearance of non-characteristic moments in the same section caused by their interaction with beams and cross beams.

Within the study the nonlinear behavior of materials has been taken into consideration by changing the rigidity of elements in plate areas with big stresses in two variants, depending on their magnitude. The analysis using the finite elements method has been verified through the calibration of the utilized model based on the results of previous tests and the final results are presented and debated in detail.

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1. Introduction

Because it is frequently found in the Romanian network of roads, the behavior of a bridge superstructure made up of 4 prefabricated beams joined through monolith plates, built according to standard projects, has been studied.

The superstructure of the analyzed bridge consists of a concrete flooring with four simply supported beams of 33 m length, prestressed concrete prefabs.. The prefabricated beams have a T section with bulb, 1,80 m height and 1,20 m the top flange width. The core thickness is of 0,16 m and the bulb of beam's inferior side is of 0,60 m width. The beams are achieved of B500 concrete mark (C32/40) and are reinforced with 7 fascicles 44ϕ5 SBP I. The monolith plates are reinforced with steel PC52 and are achieved of B400 (C25/30) concrete mark, with 0,18 m width. The distance between the prefabricated beams is of 2,70 m, the bridge traffic road has 7,80 m width and two bracket pavements of 1,00 m. The cross beams are achieved of B400 (C25/30) concrete mark and are prestressed with two fascicles 24ϕ5 SBP I. Above the resistance structure are disposed the layers of the road, represented by the inclination concrete, with 2 cm minimal thickness and 2% inclination, 1 cm waterproofing made of bituminous aluminum foil and two layers of cast asphalt 2.5 cm each.

In order to study the stress state that can occur in the bridge plates, an analysis by means of the finite element method has been made. Using the calculation program LUSAS, a linear static calculus has been achieved. The discrete model used keeps the same geometry as the real structure (fig. 1).

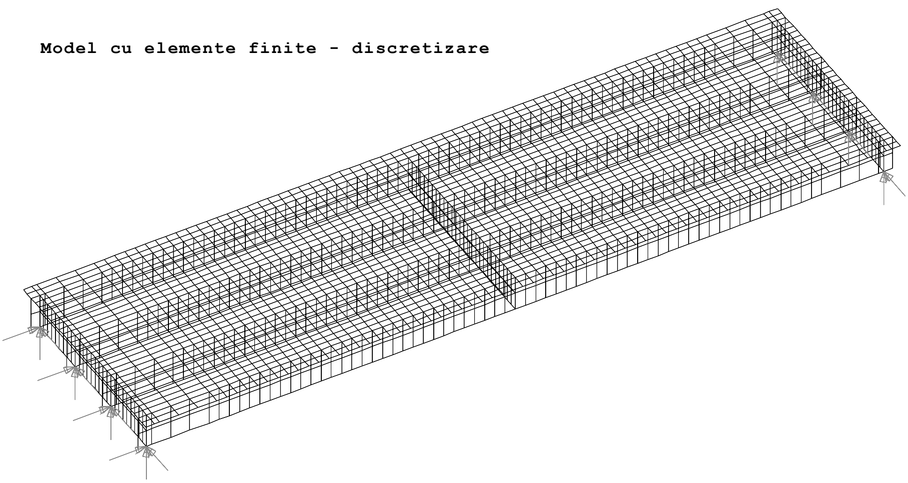

Figure 1. Model used in numerical analysis

When testing the bridge, deflections in different points of the flooring have been measured. For the calibration of the model, the aim was that the discrete structure in the LUSAS program analysis to have the most appropriate behavior to the real one. In table 1, the measured displacements and the calculated ones using the LUSAS program in beam area, in the middle of the bridge opening, under the loading utilized for bridge testing, are presented.

Table 1

Displacements (cm)	Beam 1	Beam 2	Beam 3	Beam 4
Measured	13,70	12,60	9,00	6,20
Calculated	13,70	11,57	9,10	6,44

In order to discrete the flooring component elements, two type of finite elements [3]: "shell", with both plate and QS14 type membrane behavior, with regular rectangular form and "beam", BMS3 type, have been utilized.

To discrete both monolith and prefabricated flooring plates and the cores of prefabricated beams and cross beams, elements "shell", QS14 type, with sides ratio between 1.50 and 3 have been utilized. To discrete the prefabricated beam bulbs elements "beam", BMS3 type, have been utilized. By using these types of finite elements and by choosing such dimensions for "shell", elements QSI45 type, the softest discreting of interest areas and the achievement of available results, that should not be influenced by the wrong chose of finite elements [2;4], have been taken into account.

The loadings took into consideration are the ones provided in the design existing Romanian standards and regulations for design, grouped so that the maximal stresses state in the superstructure plates should be obtained. Both, the construction technological process of the bridge and the age of concrete at the loading moment were taken into consideration.

Thus, on a model made of 4 independent beams, their own weight, the prestressing forces, the fresh concrete weight of the monolith plates and cross beams as well as the weight of the necessary casings and scaffoldings have been taken into consideration. At the geometrical model of the entire bridge it has been considered, in different loading steps, the putting out of center of the scaffoldings simultaneously with taking over by the entire structure of the monolith plates and the cross beams; the cross beams prestressing, losses of tensions in longitudinal and transversal fascicles; the contraction of plates monolith, the proper weight of permanent loadings (inclination concrete, waterproofing, cast asphalt; pavements).

The loading class for which the bridge was designed is the E loading class. Therefore this particular one has been also taken into consideration in this study. After preliminary verifications it can be concluded that the loading of bridge plates with the special vehicle V80 is more disadvantageous than loading it with truck convoys A30 [1], and that is why this loading has been used for the final study only. The vehicle position was established in order to obtain the greatest stresses in the superstructure plates. During the study, the nonlinear behavior of materials was taken into consideration by modifying the element rigidity in the plate areas with great stresses (bending moments on plates short direction).

A comparative analysis regarding the influence that the rigidity of superstructure components has on stress state in plates has been achieved for the calculus of stresses caused by the special vehicle V80. Considering two variants for modeling the material, two discrete models have been taken into account.

In the first variant we considered that the rigidity of the element is given by the elasticity module E, according to STAS 10111/2-1987 [7] and that upon the entire concrete superstructure there is an elastic, isotropic and homogenous material, with the same rigidity. Therefore it was considered:

- E1 = 36 000 N/mm^2 – for the concrete in prefabricated beams, C32/40 class;
- E2 = 32 500 N/mm^2 for the concrete in cross beams and monolith plates, C25/30 class;.

Since it is extremely difficult to model non-homogenous, anisotropic and elastic-plastic behavior of reinforced concrete for the entire bridge superstructure, the reduction of finite element rigidity in strongly stressed areas was taken into consideration.

The second finite elements model has been achieved using reduced elasticity modules, according to STAS 10111/2 – 87 provisions for the verification at limit states of normal operation (verification at limit state of strain):

- E1 = 30 600 N/mm^2 – for the concrete in cores and bulbs of prefabricated beams, C32/40 class, representing 0.85 E C32/40;
- E2 = 21 600 N/mm^2 for the concrete in plates of prefabricated beams, C32/40 class, representing 0.6 E C32/40;
- E3 = 19 500 N/mm^2 for the concrete in monolith plates, C25/30, representing 0.6 E C25/30;
- E4 = 27 625 N/mm^2 for the concrete in cross beams, C25/30 class, representing 0.85 E C25/30.

The superstructure plates are bent mainly on the short direction, on which both monolith plate elements and prefabricated plate elements behave as reinforced concrete elements. For this reason, their rigidity has been considered as 60% of the rigidity of the entire concrete section. The cores and the bulbs of beams as well as the cross beams work mainly on the prestressing direction and for this reason their rigidity has been considered as 85 % of the rigidity of the entire concrete section.

2. Acquired results

The aim was to point out the maximal stress states in the bridge plates (bending moments on their short direction) dimensioning their characteristic sections (on the support and in field), as well as the eventual appearance of non-characteristic bending moments in the same sections, caused by the interaction with beams and cross beams.

The results acquired for bending moments in plates are presented comparatively, in the diagrams from figures 2 and 3 representing the variation of moments on the support and in the field of the plate.

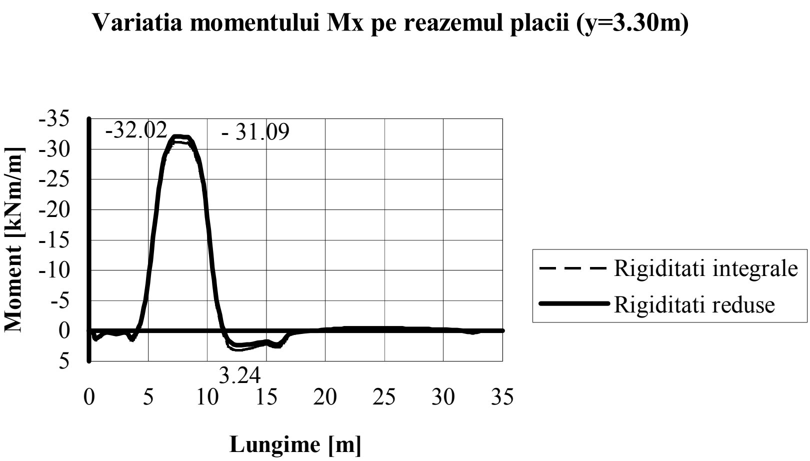

Figure 2
Variation of bending moment Mx on the support of the plate (y=3.30m)

From the point of view of stress state in the superstructure plates the differences between the two considered cases are insignificant.

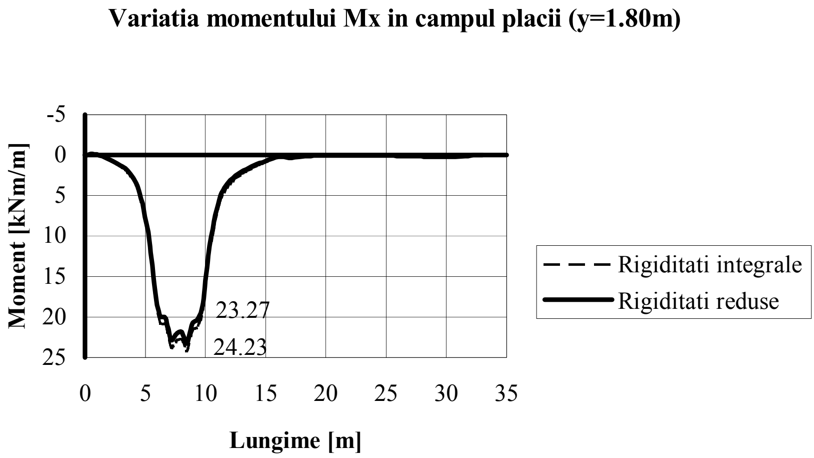

Figure 3
Variation of bending moment Mx in the field of the plate (y=1.80m)

In figures 4 and 5, the diagrams of moments in plates, in the ordinates where their maximal values are registered, are also presented.

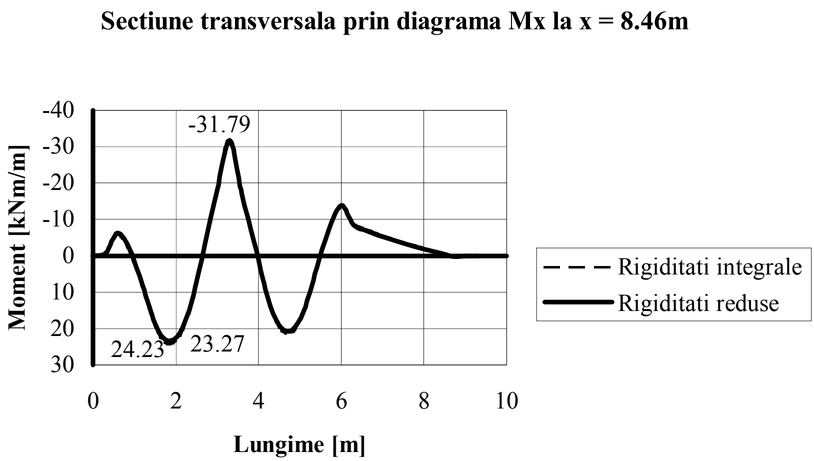

Figure 4
Diagram of bending moments Mx, for x=8.46m

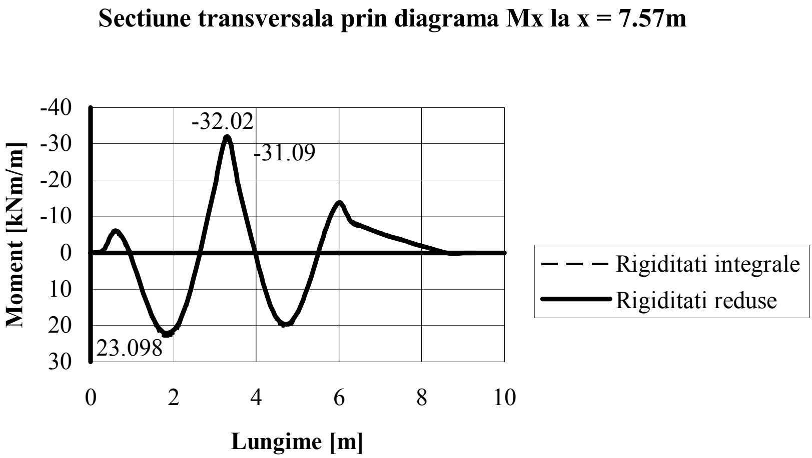

Figure 5
Diagram of bending moments Mx for x=7.57m

Thus, in the second case, an increase of 0.93KNm of the moment on plate's support, representing 3% of the global value and a reduction of 0.96KNm of the moment in the plate's field, representing 4% of the global value, are registered.

These differences appear not only by considering reduced rigidities for plates, but also by considering a reduction of rigidity different for plates and beams. Thus, while the rigidity of plates is reduced to 0.6 Eb, the rigidity of beams cores and bulbs is reduced only to 0.85 Eb.

The plate supports on the long direction, formed by the cross beams, as well as the increase of the thickness of beams core in the flooring support area, are influencing the presented diagrams by disturbing their form. Since the special vehicle V80 has been placed in the middle area of the plate panel and its influence extends for a sufficiently small length (7.60 m, meaning 23.5% from the opening of the bridge, respectively 47% of plate's length between two cross beams), in the remaining part of the plate the bending moments on its short direction have extremely small values. This phenomenon implies the fact that the disturbances produced by the cross beams and by beams core thickening are only qualitative.

Considering the analysis of the acquired results we can point out that, in the two studied variants, the major influence of a loading with the special vehicle V80 is a local one. Even in case of reducing the rigidity for the elements of the most stressed monolith plate, the participation in stresses taking over of plate areas far from the vehicle is reduced and bending moments with insignificant values (smaller than 0,42 KNm/m) appear.

3. Conclusions

Based on the numerical analyses that have been made, the conclusion is that the plates of these types of bridges, especially their middle sections, are dimensioned with a small safety coefficient and any construction deficiency could have a rather serious negative effect upon the construction behavior during the operation period.

It can also be observed that, taking into consideration reduced rigidities is practically not important from the point of view of stress state in the bridge plates, because the differences between the values obtained in these two studied cases is at most 4%.

If for a certain bridge superstructure there is no information to suggest the different reduction of rigidity, then for the calculus of the bridge plate, a most elaborated analysis by taking into consideration the reduction of its rigidity in certain areas can be avoided. This much complex approach is justified only when the real data suggest a significant reduction of rigidity in certain areas of superstructure plate.

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ESTABLISHING THE CURVES OF EQUAL VALUE OF OCTAHEDRAL SHEAR STRESS RATIO FOR ASPHALT MIXTURES FROM WEARING COURSE

Carmen RĂCĂNEL[1]

Abstract

The octahedral shear stress ratio, OSSR, a ratio between the critical octahedral shear stress induced in a pavement layer and octahedral shear strength of material, can be evaluated and analyzed from asphalt mixture rutting potential point of view.

This paper have an intention to establishing the OSSR ratio values for various depths of asphalt wearing layer (0 m, 0.005 m, 0.01 m, 0.015 m, 0.02 m, 0.025 m, 0.03 m) and for different distances by the center of applied load (0, 0.2 r, 0.3r, 0.4 r, 0.5 r, 0.6 r, 0.7 r, 0.8 r, 0.9 r, 1 r, 1.25 r, 1.5 r, 1.75 r, 2 r, where r = 0.171 m), by using the results obtained from static triaxial tests.

The calculus is based on the strains and stresses state, obtained using elastic linear software (ALIZE 5), too.

The asphalt mixtures taken into account are: asphalt concrete, BA16 and stone mastic asphalt, MASF 16, carried out with ESSO bitumen and subjected to conditions that can appear in winter, spring or summer period, in our country (3^oC, 23^oC, 40^oC temperatures).

The values resulted from calculus have finality in curves of equal values of OSSR ratio.

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1. Introduction

The octahedral shear stress ratio (OSSR) is a very important parameter which gives an indication of potential for rutting. The OSSR is the ratio of the critical induced octahedral shear stress in the pavement layer (τ_{oct}) to the octahedral shear strength of the material ($\tau_{oct,\ strength}$):

$$OSSR = \frac{\tau_{oct}}{\tau_{oct,strength}} \tag{1}$$

The speciality literature gives some results which are considered of great value and importance in the field of mechanistic analysis of pavement structures.

Research workers like Freeman and Carpenter (1986), Ameri-Gaznon and Little (1987), Perdomo (1991), Little and Youssef (1992) found that the octahedral shear stress in a pavement can indicate how close to failure a mixture is when loaded. This indication of incipient failure was given by the ratio of actual octahedral shear stress in the pavement to the failure octahedral shear stress predicted by theory.

To have a value for equation (1) we need to calculate τ_{oct} and $\tau_{oct,\ strength}$, as follows:

$$\tau_{oct} = \frac{\sqrt{2}}{3}(\sigma_1 - \sigma_3) \text{ for } \sigma_2 = \sigma_3 \tag{2}$$

where: σ_1, σ_2, σ_3 – principal stresses

$$\tau_{oct,strength} = \frac{2\sqrt{2}}{3-\sin\phi}(\sigma_{oct}\sin\phi + c\cdot\cos\phi) \tag{3}$$

σ_{oct} – octahedral normal stress:

$$\sigma_{oct} = \frac{1}{3}(\sigma_1 + 2\sigma_3) \text{ for } \sigma_2 = \sigma_3 \tag{4}$$

ϕ – angle of internal friction
c – cohesion

The shear strength parameters of material can be obtained from static triaxial test in laboratory. It is necessary to make triaxial test at failure for minimum two samples with different lateral stress.

Using an appropriate computer program (e.g. base on elastic layers theory) one can obtain reasonably good information by which to evaluate the stress state within the pavement structure under any loading.

2. Analysis

In this study two types of asphalt mixture was considered: BA16 asphalt concrete and MASF16 asphalt mixture with fiber, with 16 mm maximum size, both for wearing course, containing ESSO asphalt binder, D 50/70 pen.

From triaxial test we have the angle of internal friction and cohesion determined at 0,46 mm/min. loading rate (this means an roughly stationary load) and three test temperatures:40°C, 23°C, 3°C (table 1).

Table 1

Mixture	Temperature, °C	ϕ, °	c, MPa
BA16	40	45°51’49”	0.06129
	23	41°42’35”	0.12027
	3	38°2’1”	0.32118
MASF16	40	47°31’53”	0.15389
	23	45°55’53”	0.38347
	3	44°17’35”	0.64879

The pavement structure considered in this paper is presented in table 2.

Table 2

Mixture	h_{layer} m	Poissson ratio, μ	Elastic modulus, E MPa		
			3°C	23°C	40°C
wearing course BA16	0.03	0.35	12397	1635	266
or wearing course MASF16			12580	1717	312
existent asphalt layers	0.095	0.35	9000	2000	300
macadam	0.08	0.27	400		
soil	∞	0.35	152		

The values for elastic modulus of asphalt mixture BA16 and MASF16 were resulted from dynamic diametric test, performing in our laboratory.

The strains and stress state within above pavement structure is obtained by using elastic linear software, ALIZE 5, under standard load of our country (115 kN).

The calculus was made at various depths of asphalt wearing layer, z (0 m, 0.005 m, 0.01 m, 0.015 m, 0.02 m, 0.025 m, 0.03 m) and at different distances by the center of applied load, x (0, 0.2 r, 0.3r, 0.4 r, 0.5 r, 0.6 r, 0.7 r, 0.8 r, 0.9 r, 1 r, 1.25 r, 1.5 r, 1.75 r, 2 r, where r = 0.171 m).

3. Results

Base on those presented above we can calculate the octahedral shear stress ratio (OSSR) for asphalt mixture from wearing course. With these values of OSSR was possible to develop a series of charts showing the variation of the octahedral shear stress ratio for the two different mixes and for the three temperatures presented (figure 1 - 6).

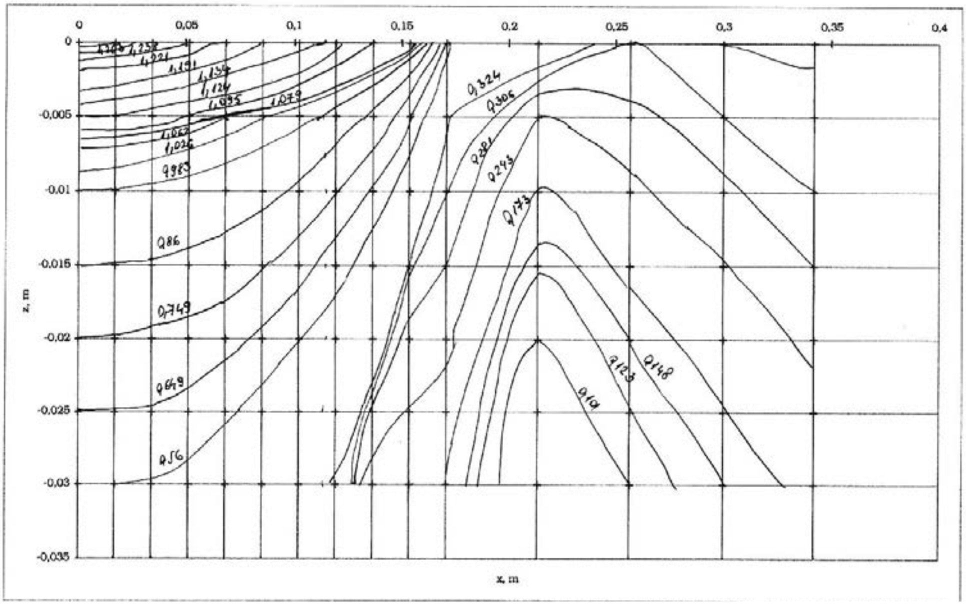

Figure 1. Curves of equal values of OSSR for BA16 asphalt mixture at 40^{o}C

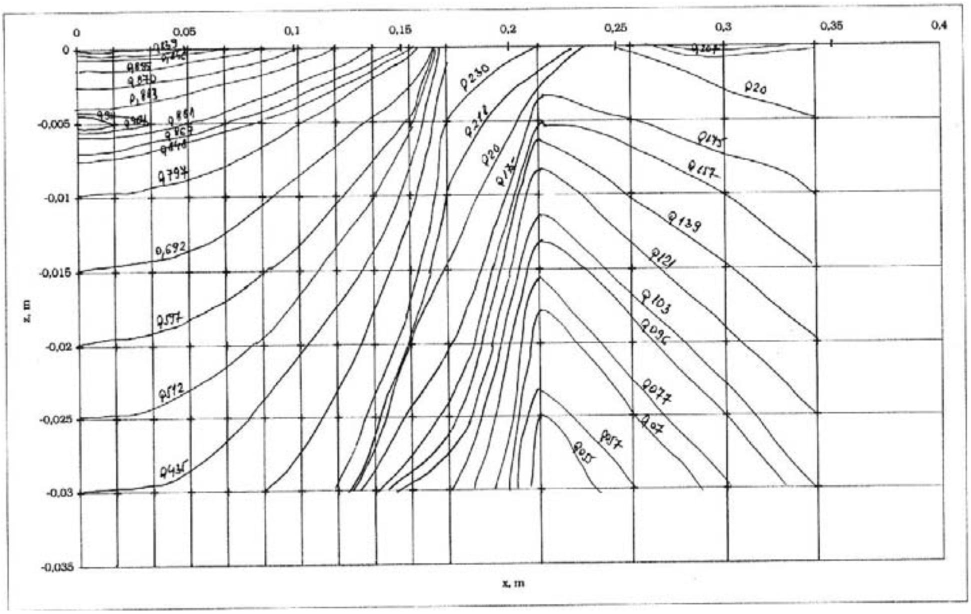

Figure 2. Curves of equal values of OSSR for MASF16 asphalt mixture at 40^{o}C

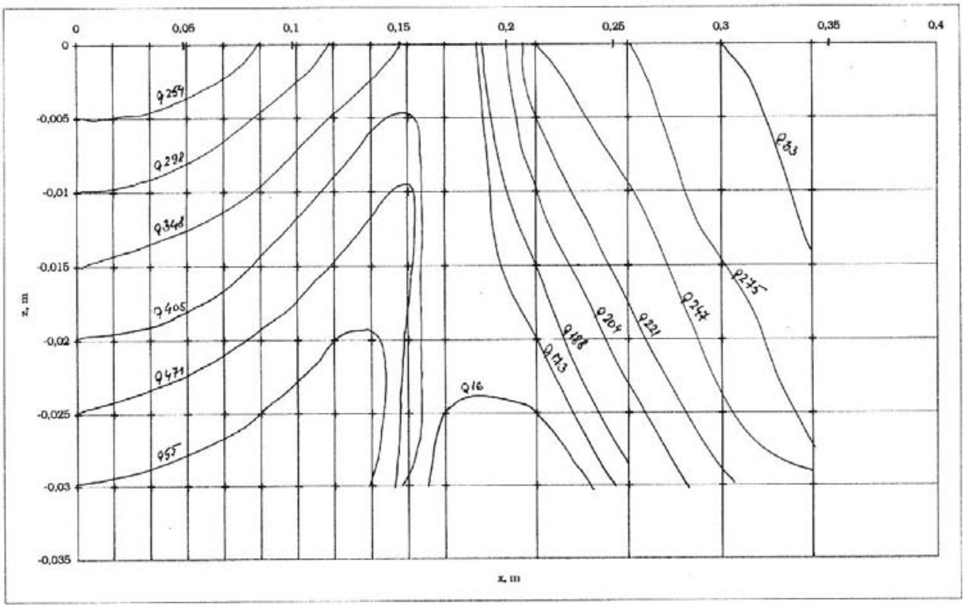

Figure 3. Curves of equal values of OSSR for BA16 asphalt mixture at 23°C

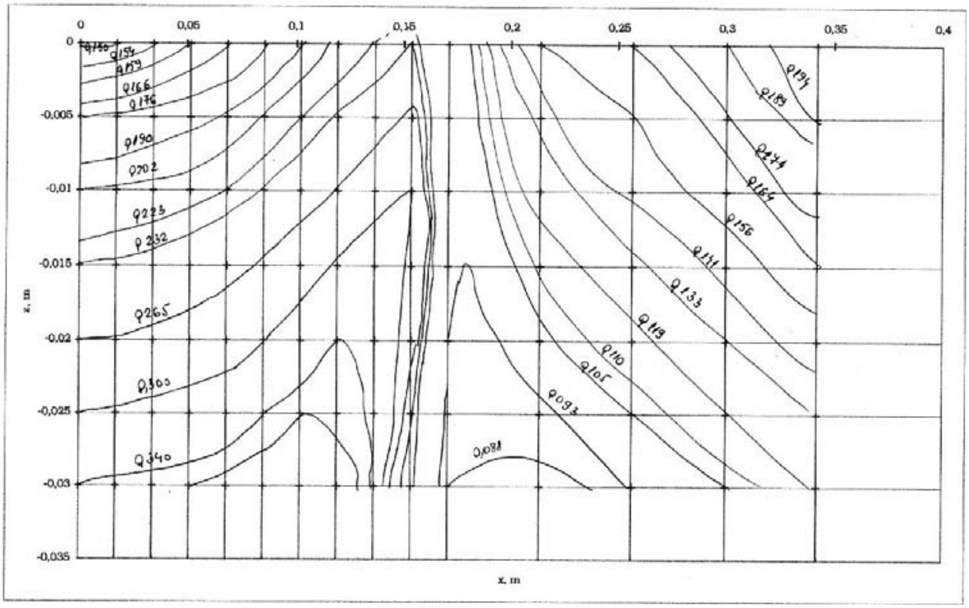

Figure 4. Curves of equal values of OSSR for MASF16 asphalt mixture at 23°C

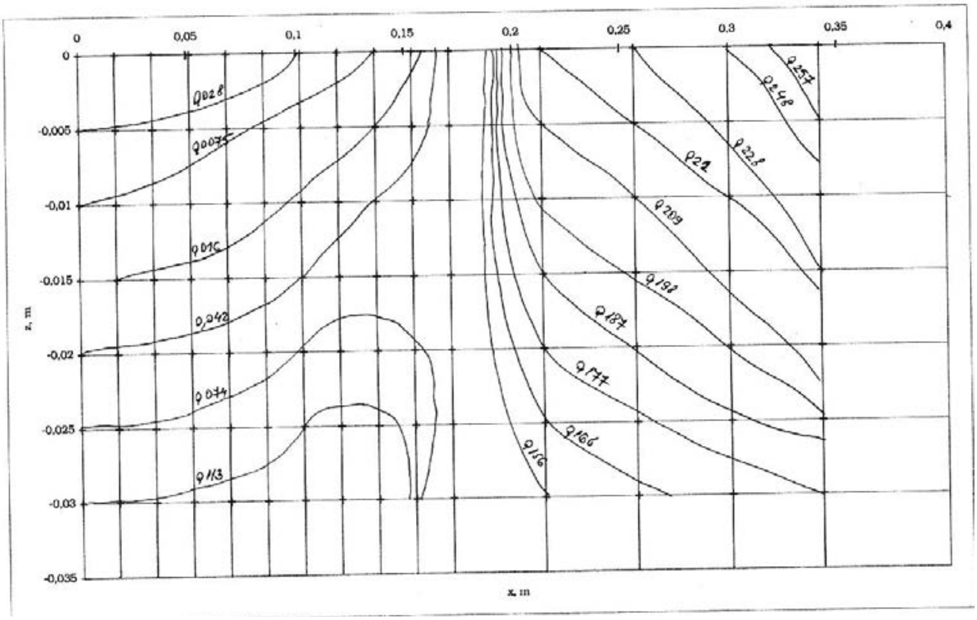

Figure 5. Curves of equal values of OSSR for BA16 asphalt mixture at 3°C

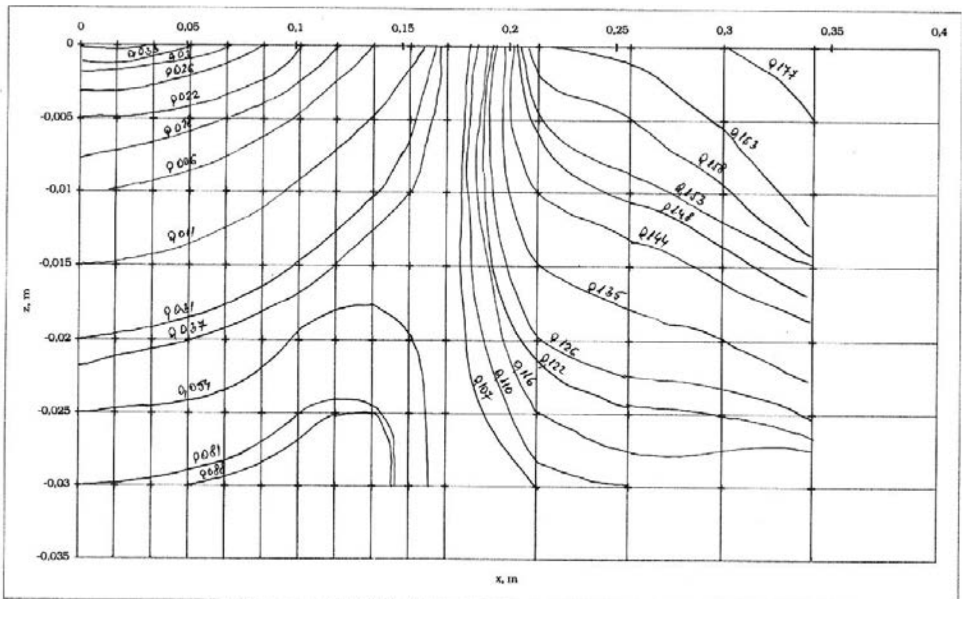

Figure 6. Curves of equal values of OSSR for MASF16 asphalt mixture at 3°C

OSSR values can be represented according to depth asphalt layer for each distance by the center of applied load. For example, figure 7 and 8 shows the OSSR versus depth layer, z in the case of BA16 asphalt concrete and MASF16 asphalt mixture, when the pavement is under summer period.

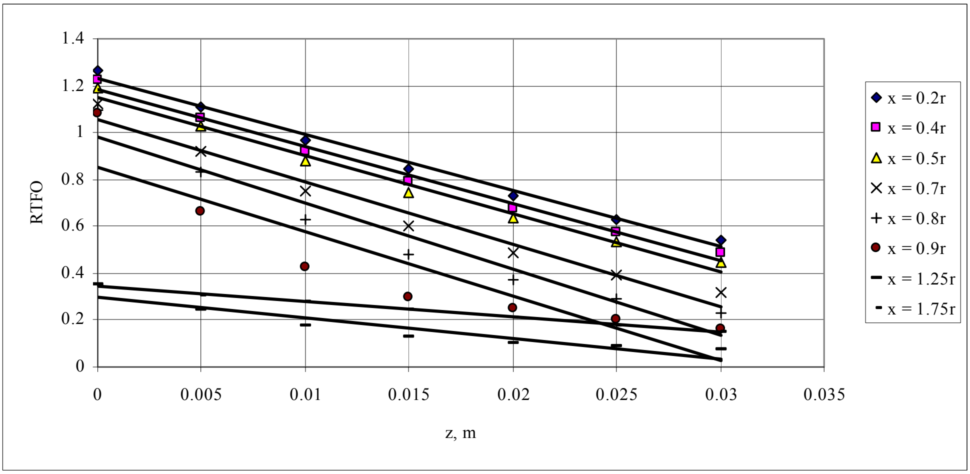

Figure 7. OSSR versus z, BA16 mixture, t = 40°C

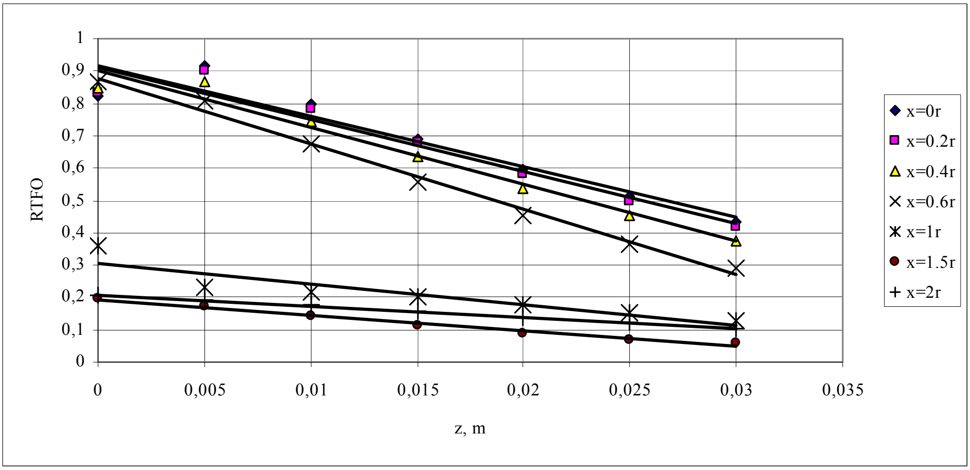

Figure 8. OSSR versus z, MASF16 mixture, t = 40°C

4. Conclusions

From this research the following conclusions can be drawn:

- octahedral shear stress ratio contours can illustrate a new way of representing failure potential within a particular pavement structure;

- for the hottest season (t = 40°C) it is observed that the potential for failure is 2,32 and 2,43 times higher than it is for spring or autumn season (t = 23°C) in the case of BA16 in wearing course and MASf16 respectively;
- for the cold season (t = 3°C) it is observed that the potential for failure is 11,3 and 10,18 times lower than it is for the hottest season (40°C) in the case of BA16 in wearing course and MASF16 respectively. This is according to well known statement that 80 percent (and more) of the permanent deformation in an asphalt concrete mixture occurs during the hottest periods of the year;
- at t = 40°C the asphalt mixture BA16 has an OSSR value up to 1 (1,2773). This signifies that octahedral shear strength of material in the layer is exceeded;
- in the case of BA16 mixture, the potential for rutting is 1,5 times higher then it is for MASF16 mixture with fiber. This should lead to the conclusion that a mixture without fiber is much more susceptible to rutting than a mixture with fiber;
- the octahedral shear stress ratio, OSSR according to depth asphalt layer, z have a linear correlation with a good coefficient, $R^2 > 0,9$, irrespective of mixture.

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RECTANGULAR HOLLOW SECTIONS – BENDING FATIGUE TESTS AND FINITE ELEMENT MODELS

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Abstract

The cold formed rectangular hollow sections (RHS) are used frequently to build the joints for the modern steel structures (buildings, steel bridges, offshores) and especially for the structural joints made of hollow sections with large width ratios b_1/b_0 it is often necessary to weld in the areas close to the edges.

Welding in cold formed areas is not only an open question for statically loaded sections (brittle fracture) but also for dynamically loaded structures (fatigue resistance). A lot of structures presented above are subjected to dynamic loadings and unfortunately a very small number of informations about the influence of welding in the cold formed areas on fatigue resistance are available.

In order to establish the fatigue behaviour of welded and non-welded rectangular hollow sections specimens, a large number of 4-point bending tests are performed at the University Friedericiana of Karlsruhe, Laboratory for Steel, Timber and Masonry.

Because these tests are very expansive, and because of the large number of parameters regarding the RHS to be investigated (steel grade, wall thickness, load distribution, boundary conditions), a number of finite element models for these specimens are made. By modeling the specimens with different types of finite elements (shells, solids) and taking into account some types of loading distributions and boundary conditions, the obtained results (stresses, strains, displacements) from a static analysis are compared with those obtained from the real 4-points bending tests.

In this way, the obtained conclusions would lead to decrease of the number of the specimens need to be tested and in the same time of the costs of the project.

Simultaneous, by using finite element analysis, many factors with direct influence on the fatigue resistance can be considered.

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1. Introduction

In the second period of the sixties, several studies and verifications of the steel structures have had in object the hollow sections, their most important characteristics and also the joining techniques, that today their use has become a current practice.

Thanks their complete technique, the hollow sections have become a very important structural element, both for engineers and architects. About 150 years already, the first rectangular hollow sections were used for the design of Britannia Railway Bridge and the first elliptical hollow sections for the Saltash railway bridge. This was the beginning in the history of the steel constructions made of hollow sections.

40 years later was built the "Firth of Forth" railway bridge at Edinburgh, like very impressive example for the use of the first circular hollow sections.

At the beginning of, the first hollow sections were made of riveted or bolted plates and angles. With the development of the continuous rolling technique and the introduction of the steel welding techniques in the 20's of the last century, the manufacturing process of the rolled or welded hollow sections has become an economical one and these structural elements have started to be used in the industry.

Welding in cold formed areas is a very sensitive problem especially for the dynamically loaded structures. Therefore, systematically gained knowledge about the fatigue behaviour of such structures is of great importance. First tests on this problem have been carried out at the University of Karlsruhe in the 1980's. During these tests, circular hollow sections (CHS) braces flattened at their ends have been welded to the corners of cold-formed rectangular hollow sections (RHS) chords, forming a K-joint. The results led to the conclusion, that in the case of high material quality, welding in cold-formed areas does not necessarily have a bad influence on the fatigue resistance.

But all kind of tests are unfortunately very expansive and in order to have reasonable conclusions, several specimens must be tested. For this reason, with the development of the computational systems and finite element programs, the design of the structures consisting of different types of sections has become easier. The best solution is always achieved by the completion of the finite element analysis with the tests carried out in the laboratory.

The purpose of this paper is to present a comparison between some fatigue tests carried out at University Friedericiana of Karlsruhe, Laboratory for Steel, Timber and Masonry and the finite element analyses, using of different types of finite elements, loading and bearing conditions.

2. Description of the performed tests and finite element models

The fatigue tests are carried out as 4-point-bending-tests (figure 1), both on welded and non-welded specimens for comparison. For all static tests, the same material quality, wall thickness and chemical composition were used. The following steel grades have been chosen:

S275J2H (Non-alloy structural steel for hollow section) as per DIN EN 10219 (Edition 1997) with the following restrictions: Content of Al $\geq$ 0.20%, S$\leq$ 0.012%, P$\leq$0.025%

S355J2H (Non-alloy structural steel for hollow section) as per DIN EN 10219 (Edition 1997) with the following restrictions: Content of Al $\geq$ 0.20%, S$\leq$ 0.008%, P$\leq$0.025%

S460MLH (Thermomechanically rolled weldable fine grain structural steel for hollow section) as per DIN EN 10219 (Edition 1997) with the following restrictions: Content of Al $\geq$ 0.20%, S$\leq$ 0.008%, P$\leq$0.025%.

The test specimens chosen were rectangular hollow sections (RHS). Depending on the availability on the market, the specimens have had different sizes, wall thicknesses and corner radii. The general dimensions of test specimens are shown, as example, in Table 1.

Table 1 Dimensions of test specimens

$b \times h \times t$ [mm]
100 × 100 × 5
100 × 100 × 8
100 × 100 × 10
100 × 100 × 12.5

All the tests are performed with the test machines from the Laboratory for Steel, Timber and Masonry at the University of Karlsruhe. First, all the specimens are measured and cleaned. A strain gage is applied to each specimen, on the tension side, on the edge in order to measure the values of the longitudinal stresses and strains. The position of this strain gage was on the mid length of the hollow section as shown in the figure 2.

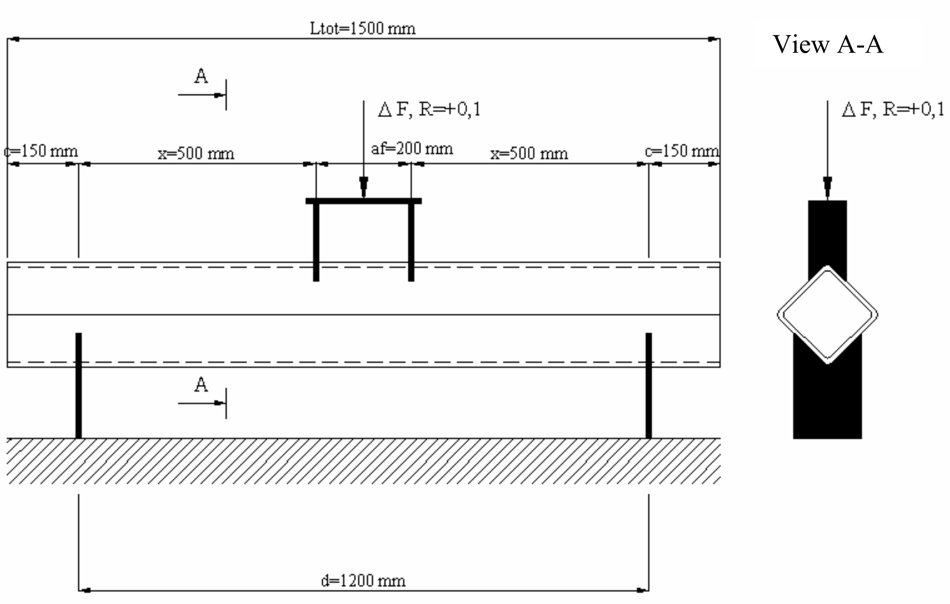


Figure 1: Scheme of the performed 4-point-bending-tests

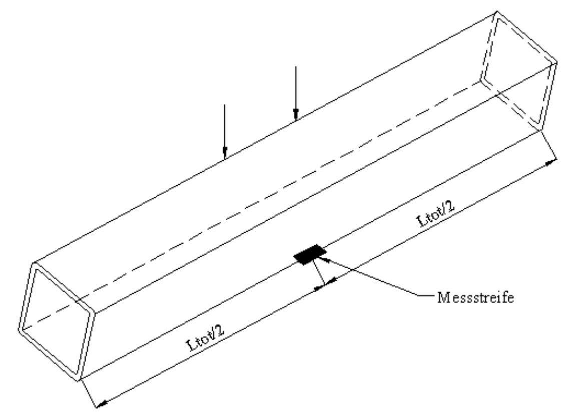


Figure 2: The position of the stress and strain gage on the test specimens

In the first stage the load was applied on the specimen in steps of 2kN until the maximal value of the load of 10kN was reached. The position of the load and also of the saddles which sustain the specimens can be seen in Figure 1, of course with different values for c, x and d for each specimen. After reaching value 10kN of the load, the process is repeated in reverse direction, also in steps of 2kN. The reason of this operation was the calibration of the machine and for the measuring software and also the remove of residual stresses in the specimen.

Following the load is applied again from 0 up to the maximum value of 10kN and back to 0, but this time the longitudinal strains are measured. The values for the longitudinal strains were:

- for a 150×150×5 hollow section: $\varepsilon_z = 0.098\times10^{-3}$ [μm/m],
- for a 100×100×5 hollow section: $\varepsilon_z = 0.279\times10^{-3}$ [μm/m]

In the same way also the stresses are established.

- for the 150×150×5 hollow section: σ_z = 20 N/mm^2. The theoretical value for the longitudinal stresses, determined by a hand calculation, depending on the chosen simplified static scheme, was of 27 N/mm^2.
- for the 100×100×5 hollow section, the theoretical value for the longitudinal stresses, determined by a hand calculation was, of 65 N/mm^2.

These values have a very large variation spectrum according to the specimen dimensions, loading and bearing conditions. Because the distribution of stresses and strains on the cross section of the hollow section and also the deformed shape of the specimen is hard to foresee, it was concluded that a finite element program can help to solve these problems.

The finite element program ANSYS was chosen. With this computer program several types of finite elements can be chosen and several bearing and loading conditions can be simulated. ANSYS consists in calculation modules and integrated pre- and postprocessors for the data input and results presentation respectively.

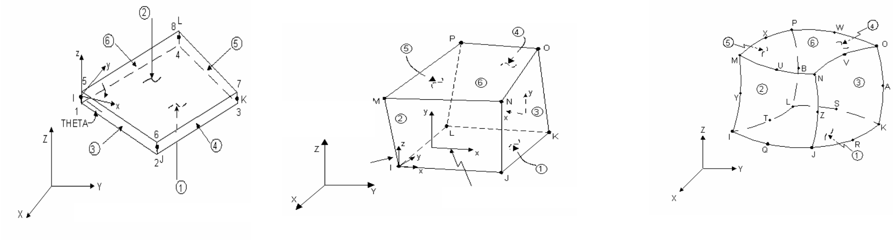
From the finite element library three kinds of elements are chosen and presented as follows.

The **SHELL63** element has both bending and membrane capabilities. He can support normal loads but also the increase of loading in several steps. This element has 6 degrees of freedom, displacement and rotations at each node.

The **SOLID45** element is used for 3D discrete models of rigid structures. The element geometry is through eight nodes defined, each node having three degrees of freedom, displacements at each node. This element can support different types of analyses: plastic analysis, creep analysis, geometrical nonlinear analysis and can be used for large displacements, large bending problems.

SOLID95 is a high order formulation of element SOLID45. It can be used to model irregular domains without significant lost of accuracy and is recommended for curved shapes. The element has 20 nodes with three degrees of freedom each, these being the displacements. This element can support different types of analyses: plastic analysis, creep analysis, geometrical nonlinear analysis and can be used for large displacements, large bending problems.

The geometry, nodes position and local axes system for these elements can be see in Figure3.



SHELL 63 SOLID45 SOLID95

Figure 3: Finite element types used in the analyses

For all specimens the finite models are created according to the conditions for the shape of the element, to the bearing conditions and distribution of loads. In the region of the specimen where the bearings and loads are placed, the finite element mesh was chosen very fine. For all the models using solid finite elements, at the beginning only one element on the thickness of the specimen was considered. After several analyses, the appropriate number of finite elements on the thickness of the specimen was established and this was kept unchanged for the rest of analyses. With help of the finite element analyses, the longitudinal and equivalent stresses and strains distribution was established. A discrete model of the specimen and the distribution of longitudinal strains are shown in figure 4

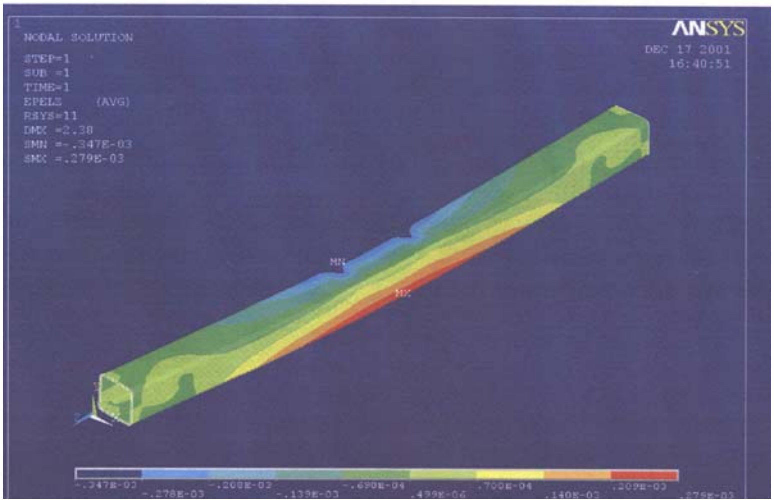


Figure 4 – The finite element model of the specimen and the distribution of longitudinal strains, ε_z

For all models, the behaviour of the material was considered linear elastic.

In Table 2 are presented, for comparison, the values of the longitudinal stresses (σ_z), for longitudinal strains (ε_z) and also for the displacement (dy) in the middle of the specimen at the bottom part.

Table 2 – The values for σ_z, ε_z and dy using different types of finite elements

Type of finite element		***ε_z [mm]***	***σ_z [N/mm^2]***	***dy [mm]***
SHELL63		0.248×10^{-3}	44.217	0.65
SOLID45	Inside	0.2577×10^{-3}	62.897	0.73083
	Outside	0.2786×10^{-3}	51.485	
SOLID95	Inside	0.2577×10^{-3}	64.362	0.73050
	Outside	0.2787×10^{-3}	52.073	
Hand calculation		0.311×10^{-3}	65.23	-
From tests		0.279×10^{-3}	-	0.76

3. Influence of the load distribution on the values of the stresses and strains

Because in the test machine, the load is applied through a saddle (as shown in figure 1) and the bearings are also represented by two saddles, in the finite element models this was modeled by taking different distribution of loads and bearing surface outside of the specimen cross section. The values for the stresses and strains are obtained in the interpolation points of the finite elements. For the models only the SOLID45 finite elements are used. The length of the uniform distributed applied load is described through the variable ***df*** and the length of the bearing surface through the variable ***dr***. The scheme of the considered model is presented in figure 6.

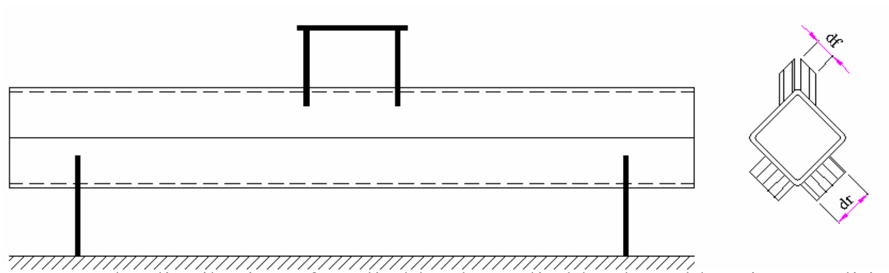


Figure 6 – The distribution of applied loads applied loads and bearing conditions

The values for ***df*** and ***dr*** measured in the laboratory were: ***df*** = 39 mm; ***dr*** = 35 mm. By taking into account different other values for ***df*** and ***dr*** the values for the stresses (σ_z), strains (ε_z)and displacements (dy) are given in Table 3. The values for stresses and strains are measured inside and also outside the finite element placed in the middle of the model, in tension.

Table 3 – Influence of the loads and bearing conditions on the values of σ_z, ε_z and dy

Type of finite element		ε_z ***[mm]***	ε_{eqv} ***[mm]***	σ_z ***[N/mm²]***	***dy [mm]***
SOLID45 df=39mm dr=35mm	Inside	$0.2577x10^{-3}$	$0.32633x10^{-3}$	62.897	0.73083
	Outside	$0.2786 x10^{-3}$	$0.41881x10^{-3}$	51.485	
SOLID45 df=70mm dr=35mm	Inside	$0.2645 x10^{-3}$	$0.313 x10^{-3}$	62.035	0.71777
	Outside	$0.2864 x10^{-3}$	$0.4092 x10^{-3}$	55.263	
SOLID45 df=39mm dr=70mm	Inside	$0.2553 x10^{-3}$	$0.3236 x10^{-3}$	62.720	0.70571
	Outside	$0.2757 x10^{-3}$	$0.4178 x10^{-3}$	50.616	
SOLID45 df=70mm dr=70mm	Inside	$0.2618 x10^{-3}$	$0.3280 x10^{-3}$	61.749	0.69188

4. Conclusions

In this paper some 4-point-bending test results and finite element models for rectangular hollow sections are presented. The type of finite element and also the modelling of external loads and bearing conditions have a strong influence on the obtained results.

Using finite elements it can be seen, that the results of the linear static analysis are close to those obtained in the laboratory (see the values given in tables above). The best results are obtained using 8-nodes finite element SOLID45, the differences between the values obtained with the computer program and the other measured in the laboratory were under 10% (for example, for a hollow section 100x100x5 with corner radius of 12 mm, the differences were: for the longitudinal strains - 1.07%, for the longitudinal stresses 1.03%, for the displacement – 3.88%.

The finite element models cannot be a substitute for the tests in laboratory, but taking into account their results, their use can lead to a decrease of the large costs of such a analysis, by reducing the number of specimens to be analysed, and also to a significant save of time.

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DESIGN PROGRAM FOR THE STABILITY OF THE JOINTLESS RAILWAY TRACK

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Abstract

The study of the jointless railway track behavior on high temperatures becomes a necessity because in the rail can appear tensions and phenomenon with spontaneous character and damaging results on the safety of the circulation.

The track works under a variety of efforts, the complexity of elements that compose the frame track, the imperfections and the non-homogenous ballast bed make difficult the correct determination both experimental and statistical of the required parameters under different methods of calculus.

The resistance of the track at transversal displacement is given by the reaction provided by the ballast bed. In case of a well maintained track, this reaction is big enough to take, without displacements, the transversal efforts the rail is normally subjected to.

The rail stability in the horizontal plane must be studied especially in curves where the tendency of buckling towards the exterior of the curve is supported by the geometry of the track itself. A compressed track will never be perfect; there will always be small eccentricities and settlement defects accentuated in time due to the action of the train movement.

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1. Introduction:

The rail stability in the horizontal plane must be studied especially in curves where the tendency of buckling towards the exterior of the curve is supported by the geometry of the track itself. A compressed track will never be perfect; there will always be small eccentricities and settlement defects accentuated in time due to the action of the train movement.

2. Calculus parameters:

The compression effort from the rail due to high temperatures can reach an appreciable value when the instability becomes a problem.
Considering the fixing temperatures interval (17°÷27°C – in Romania) and the maximum temperatures that can occur in the rail (60°C), in the central area of the jointless track can appear considerable efforts:

$$P_{\max t} = \alpha \cdot E \cdot A(60° - 17°) = 1038.45 \cdot A \tag{1}$$

$$A = 2A_s \qquad P_{\max t} = 2076.90 A_s \tag{2}$$

Heavier the rail is, higher the compression effort gets. At high effort and in certain conditions the track instability is manifested by buckling in the lowest resistance plan (in the case of modern superstructures (heavy superstructures) - the horizontal one).
The jointless railway is considered a horizontal frame, laid on the ballast bed, without being fixed and having a special geometry (alignments and spatial curves).

It is necessary to determine the calculus elements of the jointless track in order to prevent instability and to maintain the traffic safety.
The phenomenon of buckling is prevented by both the rails and the frame rail-sleeper rigidity and by the resistance of the ballast bed on the longitudinal and transversal displacement of the track.
The influence of the ballast bed has been determined with the relation:

$$q = qo + C \times y \quad \text{if } y \le y_o \text{ and } q_p = q \qquad \text{(shown in the diagram)}$$

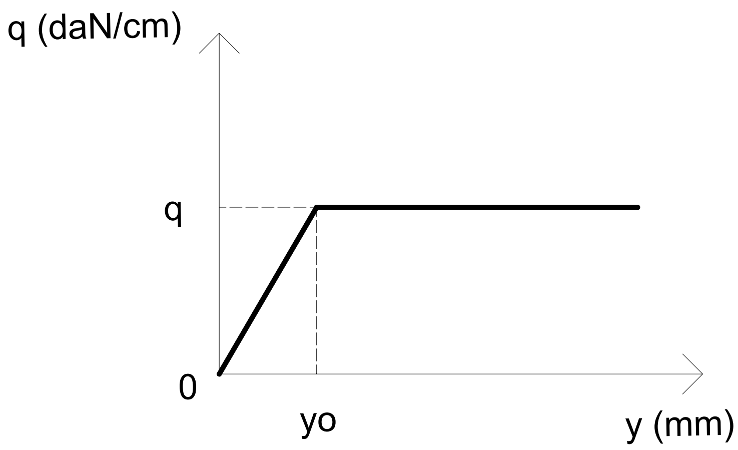


Fig.1 Lateral resistance when the displacement varies

The critical force in case of instability has been calculated using the energetic method.
For the general case (in curves), the critical force in case of instability is:

$$P_{cr} = \frac{K_1 \frac{EI}{l^2} + K_2 q_0 \frac{l^2}{f} + K_3 C l^2 + K_4 \frac{lM_r}{a \cdot f}}{1 + K_5 \frac{l^2}{fR}} \tag{3}$$

For R→∞ (alignment):

$$P_{cr} = K_1 \frac{EI}{l^2} + K_2 q_0 \frac{l^2}{f} + K_3 C l^2 + K_4 \frac{lM_r}{a \cdot f} \tag{4}$$

Where:
K1-K5 - constants for different types of imperfections;
l - length of the geometric imperfection;
f - deflection of the geometric imperfection;
C - proportionality coefficient for the ballast bed;
E - steel elasticity module;
α - steel linear thermic dilatation coefficient;

A - cross section area for the two rails; $A=2A_s$;
q - resistance of ballast bed on transversal movement of the track;
m - distributed moment; $m = \dfrac{2 \cdot M_r}{a}$
a - distance from the sleepers axe ;
r - coefficient that characterizes the fastening of the rail on the sleeper;

The equations (3) and (4) are given for five probable forms of geometric imperfections in the horizontal plane:

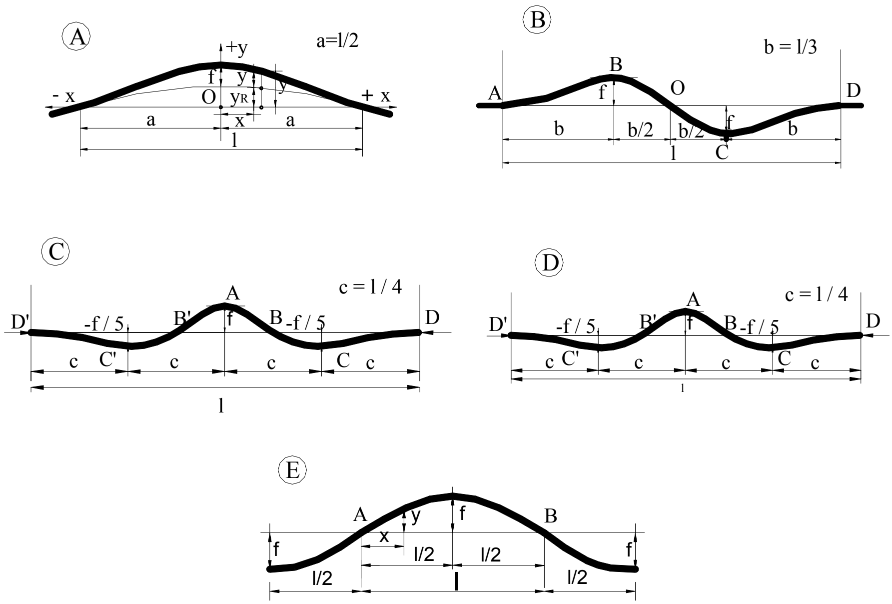


Fig. 2 Geometrical imperfections

Pcr depends on the mentioned constants and on two independent variables l and f.

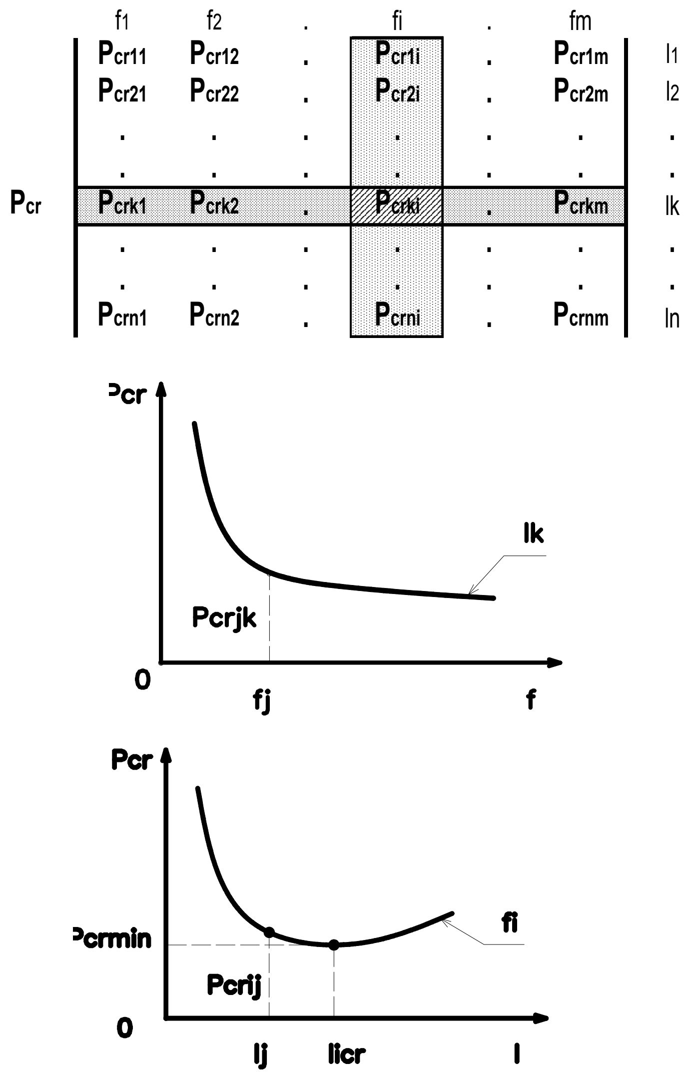


Fig. 3 The critical force matrix

where:
f= $\{f_1, f_2, \ldots f_m\}$ deflections of geometric imperfections
l= $\{l_1, l_2, \ldots l_n\}$ lengths of geometric imperfections

Example:
For the types of geometric imperfection B in alignment and A in curve the results are:

Table nr. 1
Imperfection type "B", rail type 65, r=300000 daNcm, q=6.0daN/cm, C=2.6daN/cm², qo=3.0daN/cm, a=65cm

f(cm)	0.25	0.50	0.75	1	1.5	2	2.5	3
Lcr(cm)	910.494	1043.00	1117.32	1166.58	1229.04	1267.52	1293.78	1312.89
Pcrmin (daN)	540786	414299	362205	333030	300951	283509	272489	264879

Table nr.2
Imperfection type "A", rail type 65, r=300000 daNcm, q=6.0daN/cm, C=2.6daN/cm², qo=3.0daN/cm, R=450m, a=65cm

f(cm)	0.25	0.50	0.75	1	1.5	2	2.5	3
Lcr(cm)	875.51	903.04	927.20	948.87	986.76	1019.42	1048.33	1074.38
Pcrmin (daN)	251070	236550	224854	215119	199612	187610	177909	169827

The volume of the stability calculus is high due to the variable parameters l; f and the different values of q; r and it can be reduced using a computer aided design program – presented in this paper. The program determines the necessary elements needed to analyze the jointless rail stability.

The necessary condition for keeping the rail stability even at high temperatures is:

$$P_{\max t} \leq c \cdot P_{cr\min} \tag{5}$$

c – safety coefficient, c=(1.3-1.5)

The safety coefficient and the rest of the elements are determined once the temperature starts to change. The rail temperature variation depends on the outside temperature variation. The user can set the temperature variation step, the safety coefficient is calculated and displayed with every variation. It is also displayed the case we are in: stability/instability. The initial data are the superstructure characteristics: (EI, R, q, a, Ki).
The conclusion drawn from the results provided by the program: heavier the rail is (the cross section area is bigger), the safety coefficient decreases which means that at heavy rails the critical force does not increases sufficient to compensate the increasing compression effort from the temperature.

Example:

Geometrical imperfection type B

rail type 60

$Pmax\ t_{60} < Pmaxt_{65}$
$Pcrmin_{60} > Pcrmin_{65}$

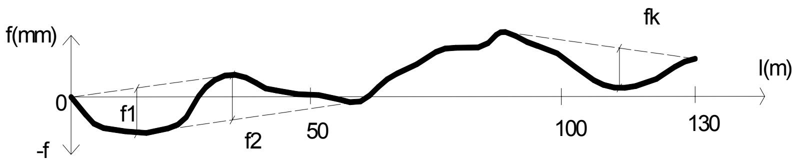
Pcrmin = 319391 daN
Pmaxt = 157532.9 daN
c = 1.31

rail type 65

Pcr min = 333030daN
Pmaxt = 171552 daN
c = 1.26

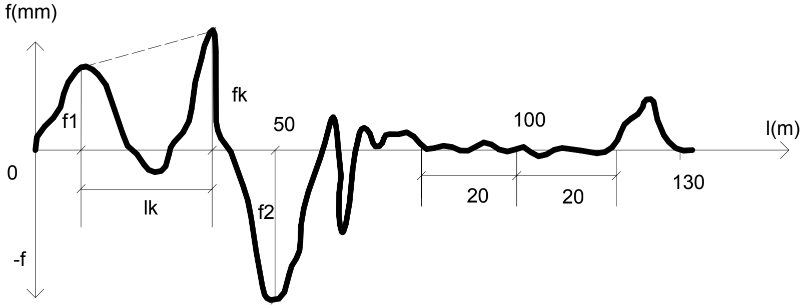
Displacement measurements of the rail in alignments:

- imperfection type: B;
- f1 = 7cm
- Pcrmin = 229600daN
- c = 1.457



Displacement measurements of the rail in curves:

- imperfection type: C;
- f5 = 8 cm;
- Pcrmin = 337700 daN;
- c = 1.457.



3. Conclusions

The critical force and all the necessary calculus ellements of the jointless rail in traffic are important in order to know the exact state of the rail and to ensure the safety in case of instability.

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International Symposium "Computational Civil Engineering"
Iasi, Romania, October 24, 2003

SPECIAL PROBLEMS CONCERNING THE ROOF STRUCTURE OF MUNICIPAL STADIUM IN CLUJ-NAPOCA

Vasile PĂCURAR[1], Ştefan I. GUȚIU[2], Cătălin MOGA[3]

Abstract

This paper presents a proposal of a roof structure for the partial covering of Cluj-Napoca Municipal Stadium tribune.

The solution has been elaborated in the fezability study regarding the rehabilitation and modernization of the stadium, according to European Union norms.

On deciding the structural solution, for the partial roofing of the stadium tribune, the following criteria have been studied:

- choosing the optimal shape for the mechanical behaviour of the structure, able to generate a minimal metallic consume (normal steel for constructions and high resistance steel for the cable stays);
- simple execution in the workshop, having selected plane elements obtained from laminate profiles and steel plates;
- subdividing the elements to obtain a minimum number of joints, at the building site;
- relatively simple joints, clearly organized on execution stages and without the risks of producing accidents on assembling the structure elements;
- safety during exploitation and easy maintenance, by avoiding the closed sections, inaccessible for cleaning and painting.

A special attention on choosing the structural solution has been paid to the aesthetic form of the building, taking into consideration that this structure besides the functional aspect – that of covering the tribune, also has an architectural function with respect to the area and the city.

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1. Introduction

The design procedure started with the documentation phase, taking into consideration other similar projects in our country and also abroad.

The models presented in Figure 1 have been selected as source (Goteborg Stadium–Sweden, Kingston Stadium – England, Vicente Calderon Stadium – Spain, Erastmus Bridge – Holland).

Goteborg Stadium – Sweden

Kingston Stadium – England

Vicente Calderon Stadium – Spain

Erastmus Bridge – Holland

Figure 1

2. Description of the selected structure

The structure selected for the covering system of the stadium tribune contains the following main elements:

- three transversal semiframes with the span of 31 m and the height of 17 m, which sustain the elements of the roof system;
- three pylons with the height of 30 m which sustain the transversal semiframes, each with one stay cable for suspension the end of the semiframe to the top of the pylon, the pylon being anchored with two inclined cables to the foundation blocks;
- the roof system contains:
    - three longitudinal main steel plate girders, spaced on 12.5 m, made up as a succession of simply supported beams with cantilever;
    - six transversal steel plate girders, spaced on 10 m;
    - truss purlins, spaced on 2.5 m;

- covering layer, made up by profiled steel sheet or polycarbonate;
- three individual foundations for the semiframes, 2x3 individual foundations for the pylons, 2x3 anchor blocks;

The assembly structure is presented in Figure 2 and Figure 3.

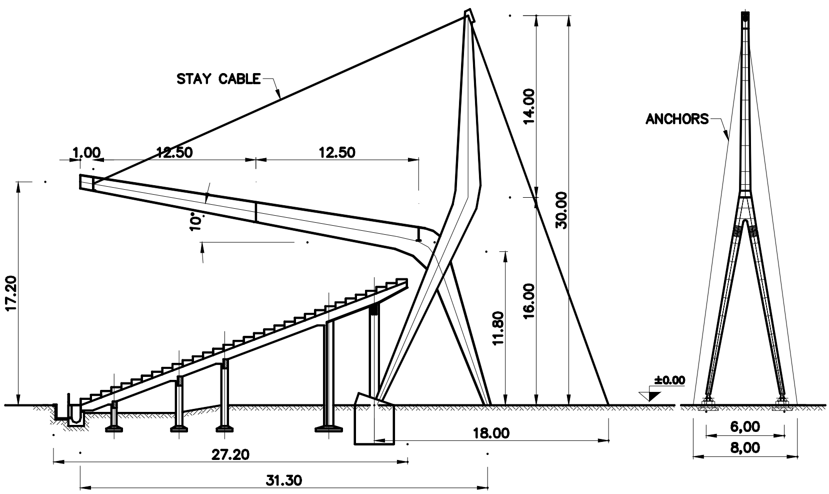


Figure 2

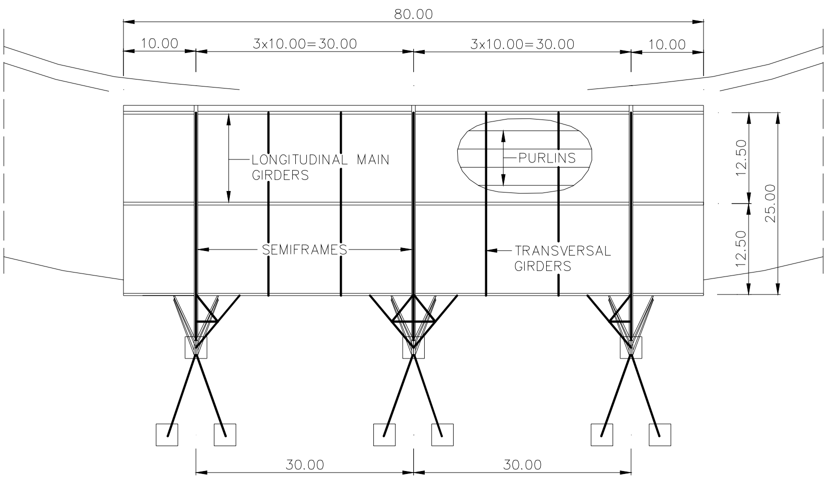


Figure 3

3. Aspects concerning the mechanic behaviour and the static calculation of the structure

The calculation of the structure has been done considering all the possible loads combinations. Taking into consideration the load combination cases the following aspects have been mainly analyzed:

- in the fundamental load combination case – the stability of the structure regarding the wind action, considering the dynamic structure characteristics;
- in the special load combination case – the structure response on seismic action.

Other considerations for the structure design are:

- the optimal shape for the semiframe, taking into account the bending moment distribution (the number and position for the stay cables), leading to the solution of a single stay cable, placed at the end of the semiframe;
- the shape of the sustaining pylon for an optimal mechanic behaviour and to obtain minimum effort in the anchoring cables;
- the structure deformations have to be situate in the allowable limits.

As follows, are present some aspects concerning the pylon conformation and the structure response on seismic action.

The pylon

Two variants of pylon building have been studied: Variant 1 - pylon fixed in foundation; Variant 2 - pylon hinged in foundation.

Each of these versions presents certain advantages and disadvantages, as follows:

- pylon fixed in foundation:

Advantages:

- o the pylon has not to be sustained in the erection stage;
- o together with the anchoring cable, it results a system static undetermined (1 degree of static undetermination), which creates certain reserves of resistance, that can be emphasized through a calculus in elasto-plastic domain.

Disadvantages:

- o at the fixing level of the pylon into the foundation a bending moment appears, thus resulting a higher solicitation of the foundation and as a consequence larger dimensions of this;
- o the fixing of the pylon into the foundation has to be ensured for undertaking of the bending moment at the fixing level.

- pylon hinged in foundation:

Advantages:

- o at the level of the hinging the pylon into the foundation the bending moment is zero, thus the solicitation of the foundation is smaller and so are its dimensions;

- o the cross section dimensions of the pylon are smaller towards the hinge level.

Disadvantages:

- o the pylon has to be sustained in the erection stage;
- o it results a determined static system, which doesn't present reserves of resistance.

From the constructive point of view in both cases have been adopted the building of the pylon with the shape of a non rectangular bar, with a vertical section in the superior half and an inclined section in the inferior half.

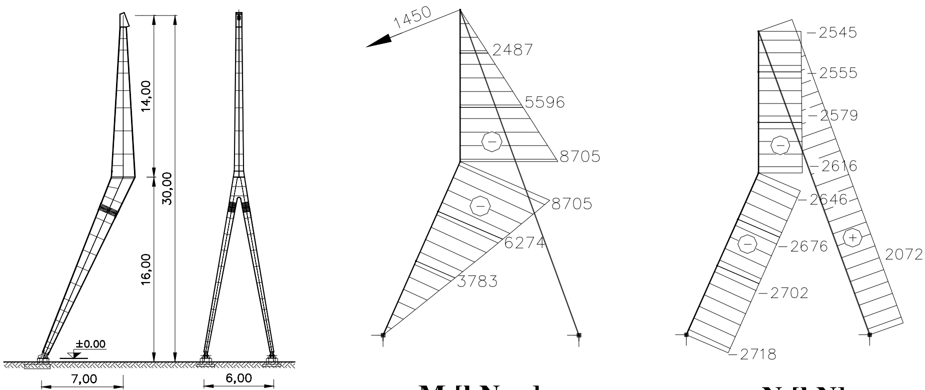
This constructive solution presents the following technical and economical advantages:

- the adoption of this shape leads to important decrease of the tension efforts into the anchoring cables, with a favorable consequence on the dimensions of the anchoring blocks and implicitly to lower costs;
- the pylon foundations can be placed closer to the existing construction, reducing the space occupied by the new one;
- the adoption of such a shape has beneficial aesthetic effects on the construction.

The inferior section of the pylon, inclined, is made by using two branches, which creates stability for the pylon in the erection and exploitation stages and much more favorable overall mechanic behaviour.

Taking into account the previously mentioned facts and the diagrams of the bending moment and axial force, it can be appreciate that the solution of the pylon hinged into the foundation (variant 2) is more advantageous than the one with the pylon fixed into the foundation.

In Figure 4 the diagrams of the bending moment (M) and axial force (N) are presented for the hinged pylon into the foundation.



M [kN m] **N [kN]**

Figure 4

The structure analyze on seismic action

The design was carried out according to the Romanian Seismic Code P100-92, the design spectrum results from the following relation:

$$c_r (= S_d) = \alpha \cdot k_s \cdot \beta_r \cdot \psi \cdot \varepsilon_1$$

$\alpha = 1.2$	- importance class II
$k_s = 0.08$	- design ground acceleration (Cluj-Napoca – Zone F)
$\beta_r = 2.5$	- spectral acceleration amplification factor, for $T_c = 1.0\,\text{sec}$
$\psi = 0.20$	- behaviour factor
$\varepsilon_1 = 1.0$	- equivalence coefficient

The analysis based on A method

The seismic analysis of any structure is being made, according to the seismic code, in a simplificated way through A method, which replaces the dynamic action of the earthquake with a static load, determined with the relation:

$$S_r = c_r \cdot G$$

where: G – in this case self weight plus snow load.

Taking into consideration the structural system (there is not a spatial cooperation between the semiframe and the pylon) the seismic load will be evaluated distinctively for the semiframe and pylon. The potential seismic loads, on semiframe and pylon, with opposite sign, are tested through load combinations.

The evaluation of seismic load for the semiframe

After evaluating the self weight (permanent action) and the snow load, it results:

$$G = \sum P_i^n + Z^n = 111080 + 199680 = 310760\,\text{daN} = 3108\,\text{kN}$$

$$S_S = 1{,}2 \cdot 0{,}08 \cdot 2{,}5 \cdot 0{,}20 \cdot 1 \cdot G = 0{,}048 \cdot G = 150\,\text{kN}$$

The evaluation of seismic load for the pylon

Self weight load of the pylon is: $G = G_{pylon} \cdot 3 \approx 23000 \cdot 3 = 69\,000\,\text{daN} = 690\,\text{kN}$

Seismic force: $S_P = 0{,}048 \cdot G = 33{,}12\,\text{kN}$

The load combination case

The calculus is made for the special load combination case:

$$\sum P_i + \sum n_i^d \cdot V_i + S$$

where:

P_i - self weight load (characteristic value);

V_i=96 daN/m^2 - variable load (here snow load – characteristic value);

$n_i^d = \gamma_1 = 0{,}35$ - long period coefficient;
S - seismic force.

For the static computation, it has been evaluated an equivalent load from the action of self weight and snow load, which is distributed to each semiframe:

$$g = \frac{\sum P_i^n + 0{,}35 \cdot Z^n}{3 \cdot L} = \frac{111080 + 0{,}35 \cdot 199680}{3 \cdot 26{,}4} = 2285\, daN/m = 22{,}9\,\text{kN/m}$$

The distribution of seismic load

- Semiframe:

The seismic force is undertaken by the three semiframes. In the static calculus the force of each semiframe is considered acting at the intersection points of the semiframe with the longitudinal beams:

$$S_{1S} = \frac{S_S}{3} = \frac{150}{3} = 50\,\text{kN}$$

The seismic force is distributed proportionally with the self weight:

- the seismic force at the contact point of the semiframe with the longitudinal boundary beam:

$$S_{GLM} = 1 \cdot \frac{S_{1S}}{4} = 12{,}5\,\text{kN}$$

- the seismic force at the contact point of the semiframe with the longitudinal central beam

$$S_{GLM} = 2 \cdot \frac{S_{1S}}{4} = 25\,\text{kN}$$

- Pylon:

$$S_{1P} = \frac{S_P}{3} = \frac{33{,}12}{3} = 11{,}04\,\text{kN} \approx 11\,\text{kN}$$

$80\% \cdot S_{1P} = 8{,}8\,\text{kN}$ - for the bottom section;

$20\% \cdot S_{1P} = 2{,}2\,\text{kN}$ - for top section.

For the static calculations the following independent loads have been established:

- **1** – self weight + snow load;
- **2** – seism action on semiframe in longitudinal direction;
- **3** – seism action on semiframe in transversal direction;
- **4** – seism action on pylon in longitudinal direction;
- **5** – seism action on pylon in transversal direction.

The following load combinations resulted are presented in Table 1:

Table 1

Load Combination	Load Cases				
	1	**2**	**3**	**4**	**5**
CO1	1	1	0	1	0
CO2	1	-1	0	-1	0
CO3	1	-1	0	1	0
CO4	1	1	0	-1	0
CO5	1	0	1	0	1
CO6	1	0	-1	0	-1
CO7	1	0	1	0	-1
CO8	1	0	-1	0	1

As a consequence of the accurate structure modeling in finite elements and the application of corresponding static forces, the efforts and the displacements of the structure have been obtained; the values resulted are inferior to those resulted in the fundamental load combination case.

Calculus based on the modal analysis (Annex C)

The seismic loads are taken into account according to the Response Spectrum Analysis method. This method requires a previously calculated number of undamped free vibration frequencies and the corresponding mode shapes.

Based on these vibration mode shapes, the software generates equivalent static loads (for each vibration mode shape) which are then applied to the model in a static analysis. Then displacement and internal force results obtained for each mode shape are summed according to the equation below, in order to obtain the internal forces:

$$Y_k = \sqrt{\sum_i Y_{ki}^2}$$

where Y_k is a cross-sectional displacement or internal force component. If two consecutive vibration frequencies are closer than 10%, the corresponding values are summed as: $Y_{j,j+1} = \left|Y_j\right| + \left|Y_{j+1}\right|$, and used later in the equation above. Due to the exponents in the equation the results give positive values. Therefore, when generating the seismic type load cases, two are included. One "+" with values included as positives, and one "-" with values included as negatives. In addition the results corresponding to each vibration mode shape are provided (corresponding to load cases with 01, 02, ….n suffixes), that can be used in the generation of further combinations or of critical combinations.

It must be specifying in addition the following parameters:

Ve - seismic wave propagation speed (Ve=400 m/s);
Lc - the largest horizontal dimension of the structure (Lc=80m).

Based on the values of the frequencies, Axis determines the β_i factors as follows (Fig. 5):

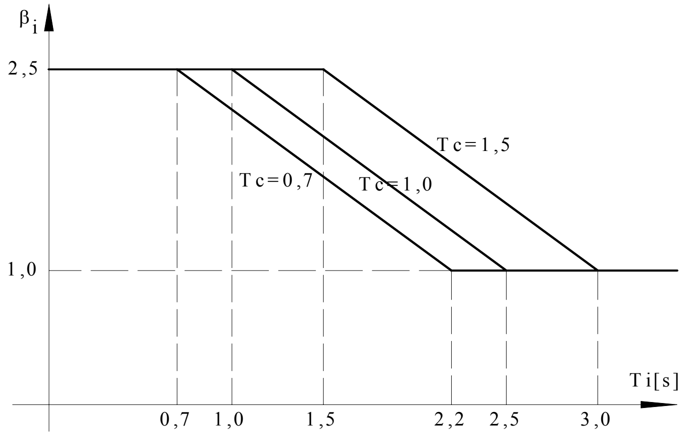


Figure 5

For each vibration mode shape, the program generates horizontal forces applying the formula below:

$$S_{k,r} = k_s \cdot \alpha \cdot \beta_r \cdot \psi \cdot \eta_{k,r} \cdot G_k$$

where:

k - degree of freedom of the node;
r - index of the respective mode shape.

In case of Romanian design code a warning message is sent if the inequality below is not satisfied: $\dfrac{L_c}{V_e \cdot T_r} < \dfrac{1}{3}$

Table 2 presents a comparison of the resulted values obtained in method A versus method C.

Table 2

		Method A	**Method C**	**Differences [%]**
Maximum effort in the stay cable	[kN]	436	408	7
Maximum effort in the anchoring cable	[kN]	263	290	9
Maximum displacement in x direction	[cm]	6,8	6,06	12
Maximum displacement in y direction	[cm]	11,4	9,32	22
Maximum displacement in z direction	[cm]	21,1	20,3	4

4. Final remarks

It can be appreciated that the solution proposed for the roof structure of the municipal stadium tribunes complies with the requests of resistance, functional and aesthetical for this specific kind of construction. Thus the construction can be integrated within the structural type category used worldwide in this field.

In what the metal requirement for the execution of the structure is concerned, the steel specific consume resulted is of approx. 80 kg/m^2, considered as having a medium value for this type of structure. If the structure of the construction is being taken into consideration omitting the pylons, the steel consume resulted would be of approx. 50 kg/m^2, which is a usual consume for the metal industrial halls with high-medium span.

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7. * * * EUROCODE 8: *Design of structures in seismic zones.*

WAVELETS SERIES ANALYSIS METHOD FOR STATIC UNDETERMINED SYSTEMS

Constantin FILIPESCU[1], **Adina RUSU**[2], **Nicu MIHALACHE**[3]

ABSTRACT. This paper presents the outcomes of using a new method for numeric calculus - wavelets functions development series for mathematical models of static stressed elements. Examples by case studies are presented in order to determine the deformations got by serial developments with Haar functions. A brief theoretical presentation of Haar type wavelet functions is inserted at the beginning. For deformations calculation Matlab tools and personal programs for differential linear and nonlinear equations solving are used.

Keywords: wavelets, bending, displacement.

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1. Introduction

The research on bending stressed elements is being done by using differential equations models. The use of wavelets function [1,2] in numeric solving of that type of problems represents a new issue that is to enlightened by simple examples that reveal the possibility of using them at more difficulty level problems.

Let's consider the exact differential equation [8]:

$$v''(x) = \pm \frac{1}{r}[1 + (v'(x))^2]^{3/2} \tag{1}$$

and the reduced one:

$$v''(x) = \pm \frac{1}{r} \tag{2}$$

Imposing certain boundary restraints to Haar series development, an expression of the deformed configuration can be determined by integration. Hence, the rotations or efforts can be determined and then the rigidity and strength can be found.

Finite or boundary element metods can be used as classical numerical solving methods.[3] Further on, a method to determine approximative solutions by series developments using Haar type wavelets functions is presented. [7] etc.

2. Haar wavelets. Elementary function integration.

In 1910 [1] A.Haar introduced an original example of orthogonal function system by which he managed to build faster transformations than those made by Walsh's or Fourier's ideas. Just after 1990 relevant applications of Haar's ideas where developed with deepering researces on the applications of some wavelets type functions in image processing and numeric calculus.

For a 4-level system [2], the integrals of Haar wavelets functions can be expres in a matrix like form as follows:

$$\int_0^1 f_0(x)dx = x \cong \tfrac{1}{8}[1\ 3\ 5\ 7] \tag{3}$$

$$\int_0^1 f_1(x) = \begin{Bmatrix} x & 0 \le x < \frac{1}{2} \\ 1 - x & \frac{1}{2} \le x < 1 \end{Bmatrix} \cong \frac{1}{8}[1\ \ 3\ \ 3\ \ 1] \tag{4}$$

$$\int_0^1 f_2(x) = \begin{Bmatrix} x & 0 \le x < \frac{1}{4} \\ \frac{1}{2} - x & \frac{1}{4} \le x < \frac{1}{2} \end{Bmatrix} \cong \frac{1}{8}[1\ \ 1\ \ 0\ \ 0] \tag{5}$$

$$\int_0^1 f_3(x) = \begin{Bmatrix} x - \frac{1}{2} & \frac{1}{2} \le x < \frac{3}{4} \\ 1 - x & \frac{3}{4} \le x < 1 \end{Bmatrix} \cong \frac{1}{8}[0\ \ 0\ \ 1\ \ 1] \tag{6}$$

A function that is square integrable on interval (0,1) (there for $\int_0^1 f^2(x)dx$ exists and is finite) can be developed in Haar wavelets series as follows:

$$f(x) = c_0 f_0(x) + c_1 f_1(x) + c_2 f_2(x) + \ldots \tag{7}$$

where

$$c_i = 2^j \int_0^1 f(x) f_i(x) dx \tag{8}$$

Let’s consider the following matriceal notation for integrals (3)-(6) :

$$\int_0^1 H_4(x)dx \cong \frac{1}{8}\begin{bmatrix} 1 & 3 & 5 & 7 \\ 1 & 3 & 3 & 1 \\ 1 & 1 & 0 & 0 \\ 0 & 0 & 1 & 1 \end{bmatrix} \tag{9}$$

Hence, it easy to prouve that:

$$\int_0^1 H_4(x)dx = P_4 H_4 \tag{10}$$

where

$$P_4 = \frac{1}{2*4}\begin{bmatrix} 4 & -2 & -1 & -1 \\ 2 & 0 & -1 & 1 \\ 1/2 & 1/2 & 0 & 0 \\ 1/2 & -1/2 & 0 & 0 \end{bmatrix} \tag{11}$$

A generally form for matrix P_m, with $m = 2^j,\ j \in N$ can be obtained using an iterative process:

$$P_m = \tfrac{1}{2m}\begin{bmatrix} 2mP_{m/2} & -H_{m/2} \\ H_{m/2}^{-1} & 0 \end{bmatrix} \tag{12}$$

Particularily, the matrix $P8$ will have the form:

$$P_8 = \frac{1}{16}\begin{bmatrix} 16P_4 & -H_4 \\ H_{m/2}^{-1} & 0 \end{bmatrix} \tag{13}$$

3. Analytical solution of differential equation (2)

Let’s choose an beam type element, being located and lean on like in figure 1.

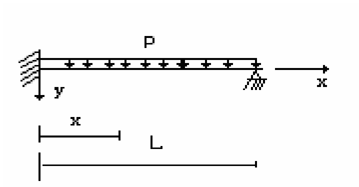


Figure 1

Initial data : L=500cm, E=2.1*10^6 daN/cm^2, b=2 cm, h=12 cm, p=2daN/cm.

Analytical solution for equation (2) corresponding to the physical model in figure 1 is obtained as follows:

$$V''''(x) = -\ p/EI \tag{14}$$

$$V'''(x) = -[-\ 5pL/8 + px]/EI \tag{15}$$

$$V''(x) = -\ [pL^2/8 - \ 5pLx/8 + px^2/2]/EI \tag{16}$$

$$V'(x) = [pL^2 x/8 - 5pLx^2/16 + px^3/6]/EI \tag{17}$$

$$V(x) = [pL^2 x^2/16 - 5pLx^3/48 + px^4/24]/EI \tag{18}$$

4. Wavelets Haar integration of differential ordinary equation (2)

The Haar wavelets like transcription of differential equation (1) is the following[6]:

$$cH(x) + \frac{1}{EI}(-pL^2 + pLx - px^2/2)(1 + [cPH(x)]^2)^{3/2} = 0 \tag{19}$$

and similarly for the reduced model (2) is :

$$cH(x) + \frac{1}{EI}(-pL^2 + pLx - px^2/2) = 0 \tag{20}$$

The Matlab solving of the two previous systems (the first – nonlinear and the second one – linear) leads to the two variants of Haar coefficients ***c***, which was used for calculating the two cases of displacements, rotation and bending moment, using the next Haar developments :

$$V = cP^2H, \quad V' = cPH, \quad V'' = cH \tag{21}$$

The following figures show the comparison between analytical and numerical Haar solutions.

It is thus objectively confirmed that neglecting the differential term within model (1) denominator may introduce only negligeable errors.

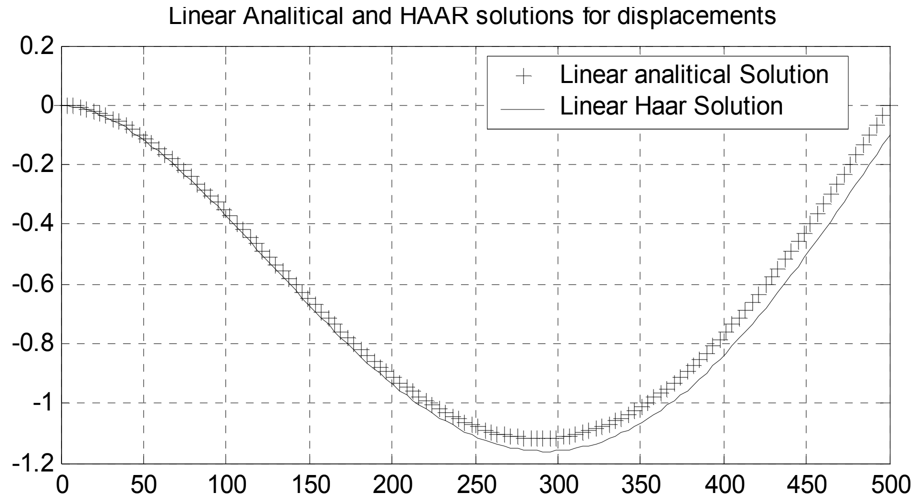


Figure 2

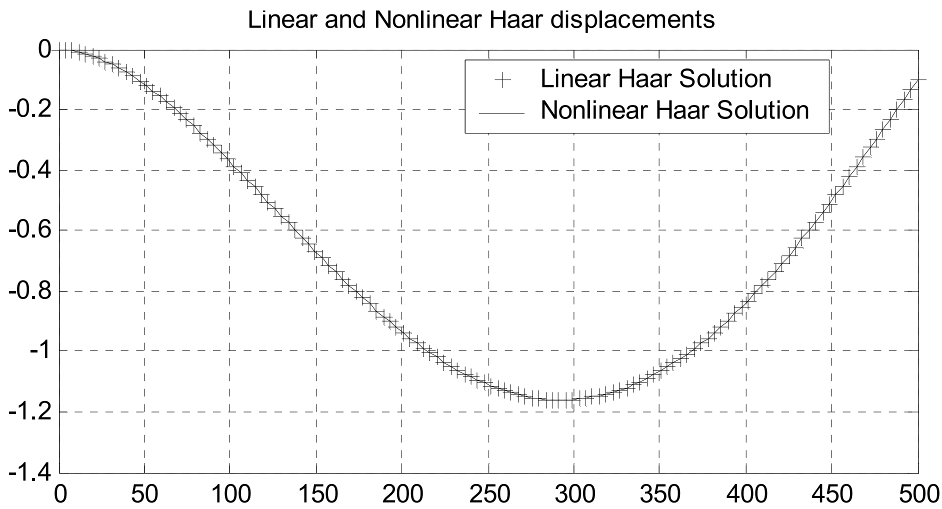


Figure 3

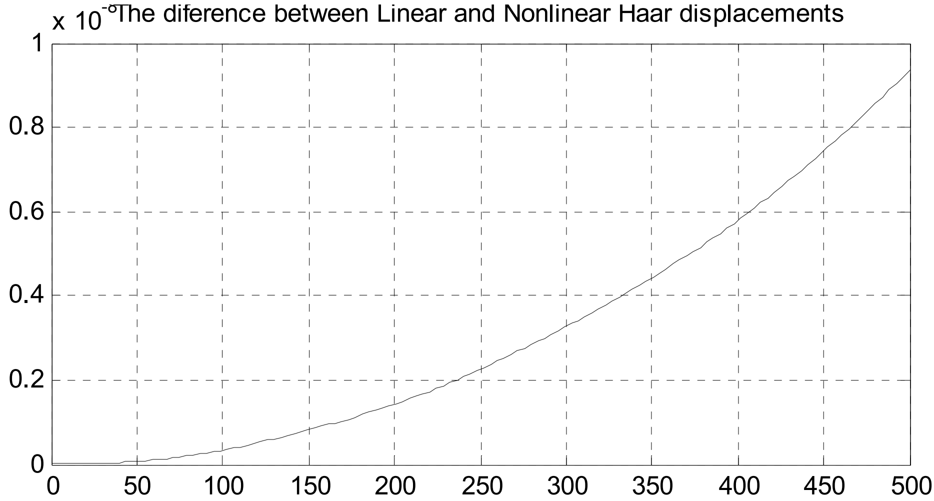
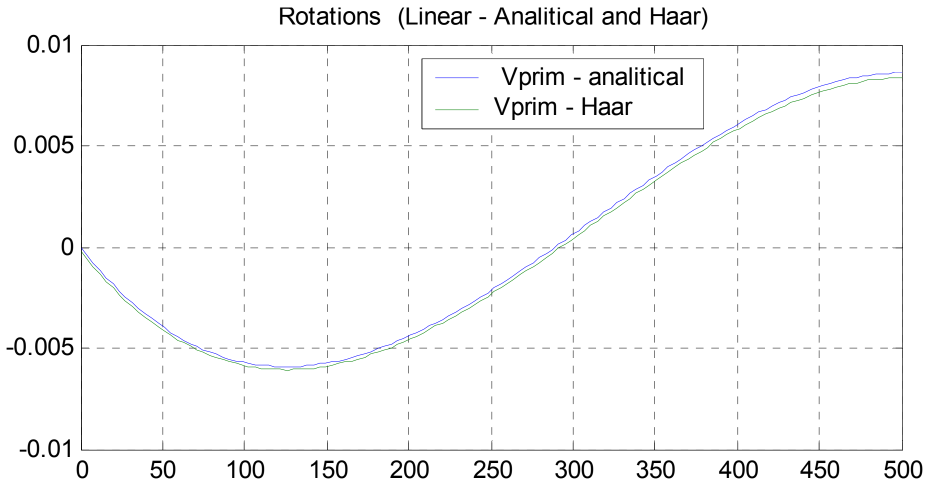


Figure 4



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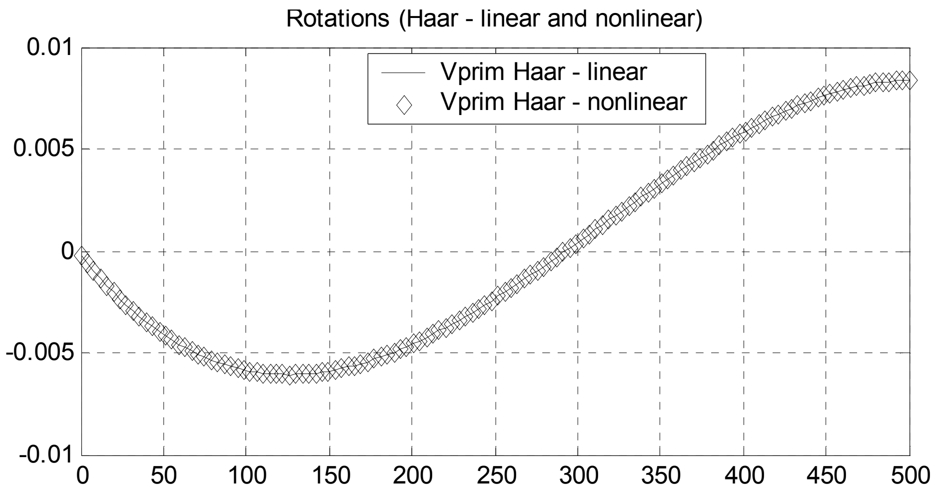

Figure 6

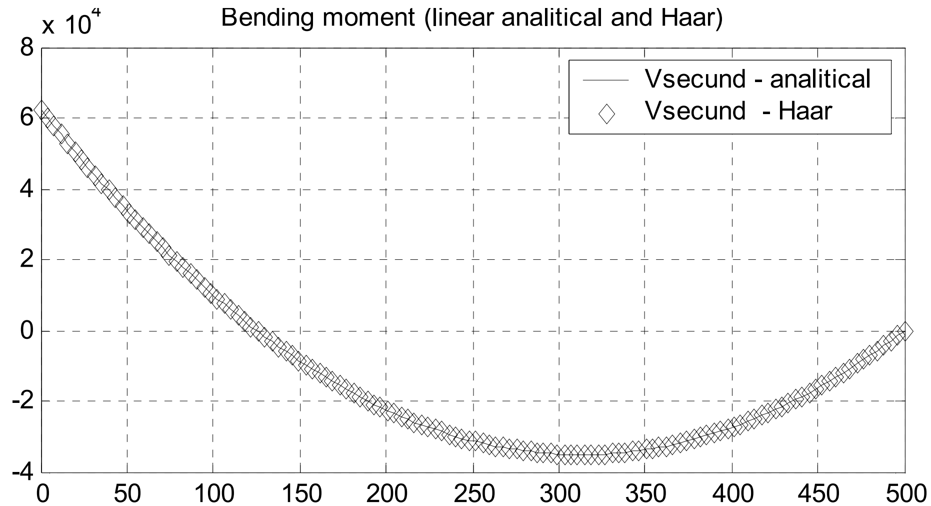

Figure 7

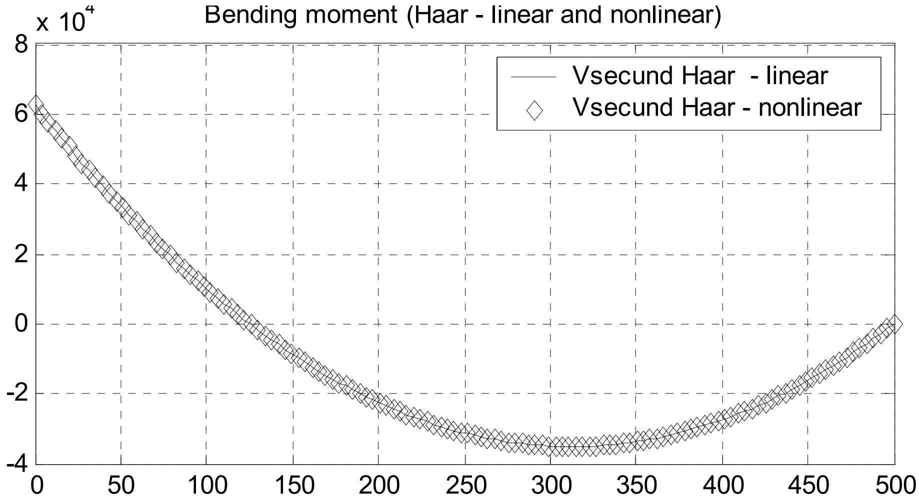

Figure 8

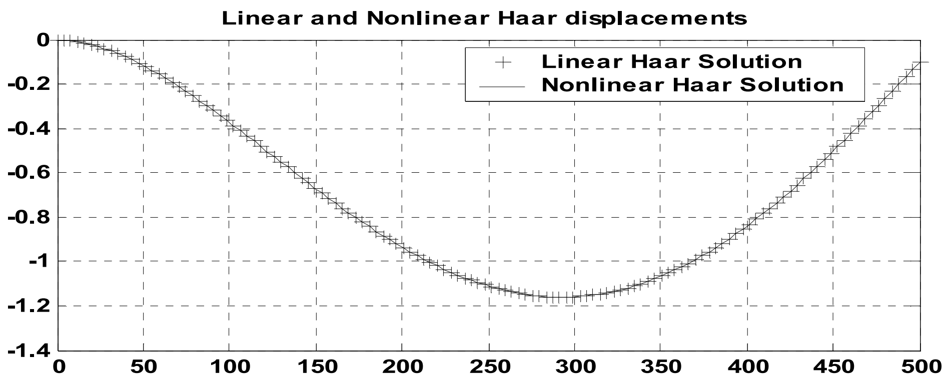

Figure 9

4. Conclusions.

Solving the differential equations through approximative methods based on developments in waveles function series allows us to get higher precision solutions. The result's accuracy and algorithm's high speed are ecouraging us in order to make some more research towards deformable solid mechanics approaches with a higer degree of complexity for wich there are no analytical solutions or they are difficult to be reached.

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ANALYS I S OF SOLID -LIQUID SEPARATION IN RAPID SAND FILTRATION

Emanoil BÂRSAN[1], **Călin IGNAT**[2]

Abstract

The water filtration is a unit process of water treatment which consists in the separation of suspensions from water by passing water through a quartz sand bed of 0.8 -1.2 m thickness.

By passing water with suspensions through porous medium, under the action of hydraulic gradient, the porous medium retains in somewhat proportions, the solid particles by different mechanisms. There takes place a mass transfer from the water subjected to filtration to the porous medium.

The clarifying process of water and the retaining the suspensions in porous medium is modeled by the following relations:

1. The kinetic equation which defines the probability of retaining the suspensions in filter medium;

2. The mass transfer equation which expresses the transfer of suspensions from the water subjected to filtration at the surface of the filter medium.

Starting from the mass transfer equations for the filtration process there were applied different solutions for the evaluation of this process.

Thus Ives (1975) used the Bessel functions of i order, and Adin and Rebhun (1977) the characteristics method for solving the coupled equations of the filtration process.

In this paper it is adapted a simplified numerical method which consists in the calculation on depth and time of the water concentration of suspensions (C) and specific deposit (σ) in terms of C and σ of the anterior steps, starting from the initial conditions (clear filter).

Also, automatically, it is realized a graphical representation of this evolution.

Because of the reduced time for the calculation and graphical representation of the mass transfer there can be analyzed o multitude of variants from which there must be chosen the rational solution for making - up the filter bed (thickness, granulation, porosity, filter surface).

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1. Introduction

The water filtration is a unit process in water treatment which consists in separation of suspensions from water by passing water through a quartz sand bed of 0.8..1.2 m thickness.

The filtration may be the unique step of water clarification or it may be preceded by the decantation operation when the content of suspensions in water subjected to filtration exceeds 15...20 mg/dm^3 (rarely and for short periods 50...100 mg/dm^3).

By passing water with suspensions through porous medium, under the action of hydraulic gradient, the porous medium retains, in somewhat proportions, the solid particles by different mechanisms [1…7]. There takes places a mass transfer from the water subjected to filtration to the porous medium.

From the modeling of transfer process it is considered that every elementary fraction from the thickness of the filter bed has a certain retaining capacity of the suspensions (Degremont, 1989) ,(Ives, 1975) .

After working up this capacity the elementary layer becomes inactive, and the retaining continues successively in the following layer up to the achieving of the retaining capacity.

The clarifying process of water and the retaining of suspensions in porous medium is modeled by the following relations (Matsui and Tambo, 1995), (Adin and Rehbun, 1977), (Ives, 1975): a) the kinetic equation which defines the probability of retaining of suspensions in filter medium; b) the mass transfer equation which expresses the transfer of suspensions from the water subjected to filtration at the surface of the filter medium.

The clarifying water phenomena and the clogging of filterable bed start from the initial conditions (clear filter) and develop during the filtration process.

They are measured by the changing produced upon the concentration of suspension (C) and the specific retaining (σ) in terms of the depth of filterable bed L and the filtration time t.

Having all the knowledge about the transport and retaining of suspension, the filtration process can't be rendered directly by physical properties of the liquid that contains suspensions and the characteristics of the porous medium.

As a result a size experimentally determined is used for interaction between the suspensions existing in water and the filter medium. This size is the filter coefficient λ (Hedberg.1976), (Ives, 1975).

The most general model which determined the filter coefficient σ is that which takes in to consideration the changes of the specific surface of pores and the increasing of the interstitial velocity due to narrowing in time of the water flowpaths in pores.

Starting from the fact that the initial porous medium may be represented by an assembly of individual spheres and finally of filtration as an assembly of individual cylindrical capillaries and taking into account the modification of interstitial velocities as a result of suspension retaining in pores, the general model of the filter coefficient has the expression (Ives, 1975).

$$\frac{\lambda}{\lambda_0}=\left(1-\frac{\sigma}{1-\varepsilon_0}\right)^x\left(1-\frac{\sigma}{\varepsilon_0}\right)^y\left(1-\frac{\sigma}{\sigma_u}\right)^z \tag{1}$$

where:

λ - the filter coefficient at time t;

λ_0 - the filter coefficient at time t = 0;

ε_0 - the initial porosity of filter medium;

σ - the specific deposit (the volume of deposit per unit filter volume);

σ_u - ultimate specific deposit;

x, y, z - the exponents expressing the influence of different terms.

The significance of model established for λ (s. relation (1)) consists in the fact that with appropriate selection of values for the exponents x, y, z may be generates many mathematical models proposed for λ of different authors.

Apart from the attached processes of particles from the porous medium it develops a process of detaching which is quantified by the detaching (scour) coefficient α that is a function of the recorded retaining (σ) and filter velocity (v).

Starting from the mass transfer equations for the filtration process there were applied different solutions for evaluation this process.

Thus in (Ives, 1975) it is used the Bessel functions of i order, and in (Adin and Rehbun,1977) the characteristics method for solving the coupled equations of the filtration process.

In this paper it is adapted a simplified numerical method which consists in the calculation on depth and time of the water concentration of suspensions (C) and specific deposit (σ) in terms of C and σ of the anterior steps.

Also, automatically, it is realized a graphical representation of this evolution.

2. Mass transfer equations in the filtration process.

If it is admitted that in the variation of suspension accumulation process the concentration is proportional to the local concentration of suspensions (Hedberg.1976) ,(Ives, 1975) , the kinetic equation has the form:

$$-\frac{\delta C}{\delta L} = \lambda C \tag{2}$$

where: C is the water concentration of suspension; L - the depth of the filter bed; λ - the filter coefficient.

At the beginning of the filtration when t = 0; $\lambda = \lambda_0$ and for L = 0 $C = C_0$ (C_0 - the concentration in suspension at the filter input).

The suspension deposit at t = 0 on the filter depth L is:

$$C = C_0 \exp(-\lambda_0 L) \tag{3}$$

The balance equation has the form (Hedberg.1976) , (Ives, 1975) :

$$-\frac{\partial C}{\partial L} = \frac{1}{v}\frac{\partial \sigma}{\partial t} \tag{4}$$

in which at the former notations there are added: v - the filtration velocity; σ - the specific retaining (deposit); t - the time.

At a certain filtration velocity a fraction from the retaining is washed and comes back in suspension, reducing the growing velocity of σ.

Considering that the detachment is proportional to specific retaining (σ) and introducing the washing (detachment) coefficient α, the retaining velocity of suspension in porous medium becomes (Hedberg.1976) , (Ives, 1975):

$$\frac{\partial \sigma}{\partial t} = \lambda v C - \alpha\sigma \tag{5}$$

Introducing (5) in (4) it results:

$$-\frac{\partial C}{\partial L} = \lambda C - \frac{\alpha}{v}\sigma \tag{6}$$

For studying the transfer process there will be taken into account the balance equation (4) and kinetic equation (6).

Because in the equation (6) there occur the coefficient λ and σ, variable in time and on the filter depth, there will be used for these coefficients the following expressions:

$$\lambda = \lambda_0\left(1 - \frac{\sigma}{\sigma_u}\right) \tag{7}$$

This expression results from (1) by appropriate selection: x = y = 0; z = 1 (Hedberg.1976). There can be adopted other expressions for the coefficient λ (Hedberg.1976), as well.

$$\alpha = \lambda \, v \frac{C_0}{\sigma_u} \tag{8}$$

Relation (8) derives from (5) after (Ives, 1975), for $\partial\sigma/\partial t = 0$. In this case $C = C_0$ and $\sigma = \sigma_u$.

Replacing (7) in (8) obtain

$$\alpha = \lambda_0 v \left(1 - \frac{\sigma}{\sigma_u}\right) \frac{C_0}{\sigma_u} \tag{8'}$$

3. Numerical Solution of Mass Transfer Equations.

For solving the equation system (4) and (6) together with (7) and (8) in the (L, t) plane, there is considered a square network of step $\Delta L = \Delta t = 0.01$ (L - filter thickness; t = time).

To the equations with partial derivatives (4) and (6) the following equations with finite differences:

$$C_{ij} = C_{i-1,j}\,(1 - \lambda_i\,\Delta L) + \frac{1}{v}\alpha_i\,\Delta L\,\sigma_{i,j-1} \tag{9}$$

$$\sigma_{i,j+1} = \sigma_{i,j}\,(1 - \alpha_i\,\Delta t) + \lambda_i\,C_{ij}\,v\,\Delta t \tag{10}$$

may be attached..

The system (9), (10) for i = 0, 1,...,[L/ΔL] and j = 0, 1,...,[t/Δt] finding first C_{ij} and with obtained values $\sigma_{i,j+1}$ with an approximation error of $\Delta L = \Delta t = 0.01$ order.

The mass transfer is followed by modifying the concentration of suspensions in filtered water C with the relation (9) and of accumulations σ from the sand bed pores with the relation (10) in terms of filter bed depth (L) and time (t).

The calculation and representation of C and σ on the filter depth and time levels were realized with the help of a program which calculates and represents graphically the concentration evolution in suspension of the water subjected to filtration (C) and specific retaining in pores (σ). The program have a general character. The values of parameters for which are done calculation are established by a dialog window that permits following limits (which can be modified if the necessity imposed):

Sand layer thickness: 0.1.…10 m; initial concentration C_0 : 1….100 mg /dm^3 ; velocity of filtration: 0.5…10 m/h; sand grain diameter; 0.1…10 mm; λ_0 = 1…10 ; σ_u = 1000 …20000; filtration run : 20…32 ore.

4. Example

It is considered:

1. A sand filter bed with the following characteristic: L = 1 m; d = 0,7 mm (d - the sand grain diameter); σ_u = 11720 mg/dm^3 (according to (1) table 2 for d = 0.7 mm and coagulant $Al_2(SO_4)_3$) ; λ_0 = 6.55 m^{-1};

2. Concentration in suspensions of the influent C_0 = 50 mg/dm_3;

3. Filtration velocity v = 5 m/h;

4. Filtration time t = 20 h;

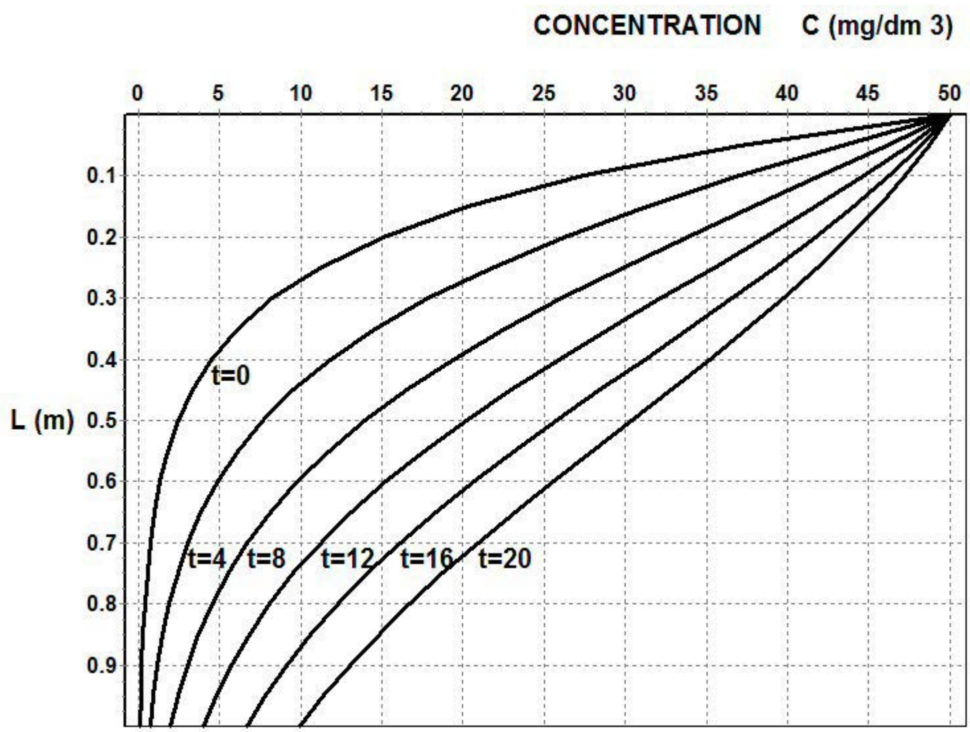

Fig.1.

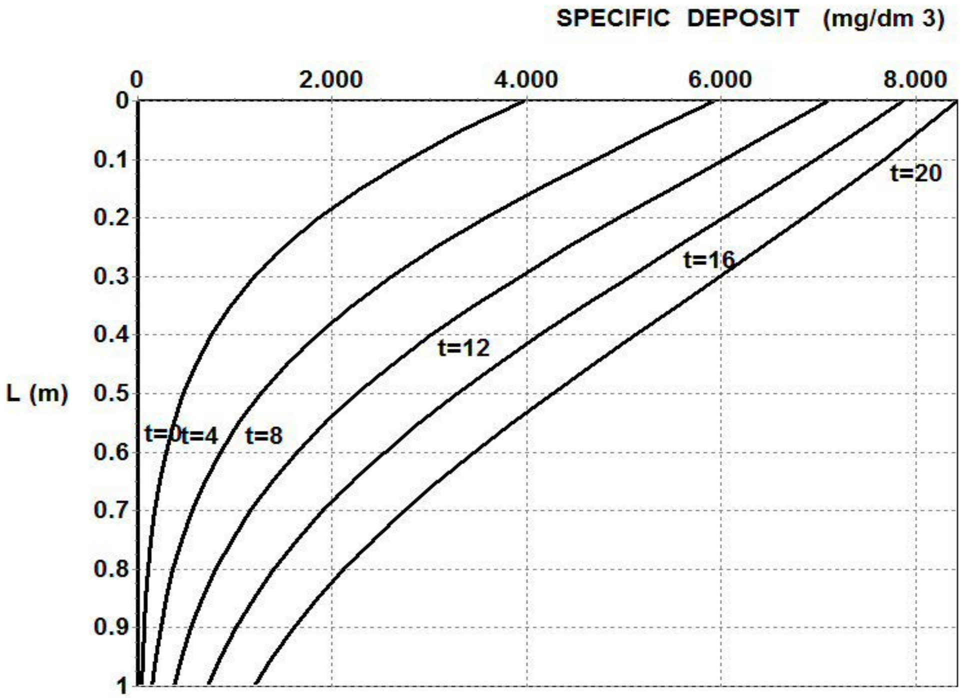

Fig. 2

In fig. 1 it was represented the variation of concentrations in suspension of water subjected to filtration in terms of depth (L = 0...1 m) and time of filtration (t = 0...20 h , graphically represented at time 4h interval) that is C = f(L ,t);

In fig. 2 it was represented the variation of specific retaining (σ) in terms of depth (L = 0...1 m) and filtration time (t = 0... 20 h, graphically represented at 4 h interval), that is σ = f(L , t)

By determining of the values for C and σ for different levels, and their graphical representations, the scaling is done by the bed thickness (L) and filtration time (t) which in the case of given example is L = 1 m and respectively t = 20 h.

By using the exposed method it is found that transfer process of suspensions is much more accentuated in the superior layers of the filter in comparison with the layers of the inferior half. This fact verifies those said in (Adin and Rehbun,1977) and (Ives, 1975) with another methods for solving the equations (4) and (6).

These findings had as a result the achievement of bi - flow filters (with descendent and upward current) where using of retaining capacity of the filter bed is done more rationally.

5. Conclusions.

1. Mass transfer in the process of water filtration with sand filter can be modeled by relation (4) and (6);
2. Having in view that in the specialized literature there are several relations for determining filter coefficients λ and α the adopting of some expression must be done by the experimental checking on a pilot filter of appropriate dimensions;
3. Knowing the filter coefficient λ and α the mass transfer calculus and its graphical representation can be realized numerically on depth and time levels with the relation (9) and (10) using an own automatic program;
4. Because of the reduced time for the calculus and graphical representation of the mass transfer there can be analyzed o multitude of variants from which there must be chosen the rational solution for making - up the filter bed (thickness, granulation, porosity, filter surface).

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EVOLUTION OF CHLORINE CONCENTRATION IN WATER DISTRIBUTION NETWORKS

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Abstract

A water distribution network has as a function the continuous supply of consumers with water of quantity, quality and service pressure.

By the hydraulic calculation of the networks there are assured the flow rates and the necessary pressures at the consumers.

The water quality is obtained by the corresponding treatments while the maintaining of water quality during the time of passing distribution network is realized by the enter diminishing in the network of biodegradable organic substances supplementing the treatment in the last phases with the ozone - granular active carbon (GAC) coupling, slowing down the bacterial development by introducing chlorine in the network with biological stability effect, and by periodical washing, in order to eliminate the settlings in zones with slow velocities.

The present paper deals with a model of following the chlorine concentration evolution during the time of network transit.

The chlorine concentration calculation at nodes and the ends of the pipelines is realized with the help of a linear algebraic system of T (pipelines) + N (nodes) equations and knowledge or imposing the kinetic constant value for the chlorine reaction.

The results of numerical modeling of quantitative (hydraulic) and quantitative (chlorine concentration) of a distribution network has a practical value only if calibrations are done for both situations, operations which are difficult to realize without computerized mapping.

The evolution of free chlorine concentration is illustrated with the help of a personal program applied to a network of large dimensions.

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1. Introduction

Before being introduced in the distribution network, water is disinfected with chlorine (primary disinfection).

Since the chlorine is a remanent disinfectant, irrespective of the way the disinfection is done, it is introduced in water that supplies the distribution network. This chlorine has a bacteriostatic property (the secondary disinfection).

In the process of maintaining the water quality the chlorine is consumed. This consumption depends on the presence of organic substances in the water distributed by the network and the time necessary for water to reach the consumer.

In Romania the Law 458-2002 concerning Drinking Water Quality requires that: at the inlet the distribution network, free residual chlorine is max. 0.5 mg/dm^3 and at the consumer (end of network) free residual chlorine is min. 0.25 mg/dm^3.

Water quality (content in organic compounds) and the time traversed by water in the network can lead to the situation when it is not realized the above specified concentration at the consumer, in other words a qualitatively corresponding water is not distributed. For establishing the causes that lead to this situation it is necessary to model the chlorine concentration evolution in the distribution networks, that is to know the chlorine concentration at network nodes (the consumption nodes inclusively) in comparison with the chlorine concentration at network inlet.

Knowing this fact helps us to chose the re-chlorination points in the network or the modification that are in view to operate for the links between the pipelines in order to avoid the chlorine concentration under the prescribed standard limits.

2. The chlorine role in maintaining the water quality in the distribution network.

After treatment, water is carried to the consumers by the distribution network made up of different materials and diameters, this network must conserve the water quality obtained in the treatment station, this being done with difficulty in some situations.

The water quality in the distribution network depends on the water treatment way in accordance with the use requirements and the network characteristics as well (length, diameters, work conditions etc).

In the distribution network the water quality change can results from: a biologic ecosystem development; the constitutive network materials influence; accidental contamination. The distribution network being an open system (reservoirs, taps, etc) there exist plenty of contamination sources that lead to the formation of an ecosystem. Unlike the natural ecosystems where the energy source is light, in the ecosystem formed in the distribution network the main energy sources are represented by organic carbon under the form of dissolved organic substance remained in water after treatment. This is the source of feeding the bacteria that develop into water (they move about freely) and on the inner walls of the pipelines under the form of bacterial biofilm (fixed bacteria). The bacteria are the starting points in a trophic chain that stops at the crustaceans (e.g..asellus aquaticus). The biologic growth base is in the main, bacteria from biofilm (A.G.H.T.M., 1990). This biofilm, very heterogeneous, comprises over one million of bacterial cells per cm^2 and is usually present in the wet natural environment. The principal food of bacteria consists of biodegradable dissolved organic carbon that assures their metabolism. These bacteria are not dangerous for man, but they can lead directly or indirectly to drinking water quality modifications (unpleasant taste, small animals etc). The prevent of bacteria proliferation is done by the choosing of a treatment chain that removes the organic carbon, chlorine disinfection and the washing away of the pipelines. The chlorine treatment at the network entrance permits the stopping of the bacterial proliferation due to the bactericide remanent effect.

Adding chlorine to a tolerated gustative concentration does not eliminate the biofilm but only limits the bacterial growth and eliminates the free bacteria in water. Even with these limitations, chlorine is an efficient way of maintaining the water quality in the network. After (A.G.H.TM, 1990) neither dose of 0.3…0.4 mg/dm^3 impede the biofilm formation on inner wall of the pipelines, but only reduced the formation velocity of biofilm and the development of a trophic chain.

3. The free chlorine consumption equation in the distribution network

The water from the distribution network chlorine reacts with numerous compounds (mineral or organic) or microorganisms (bacteria etc) occurring in water mass or on the inner surface of the pipelines.

The chlorine reactions depend on the water temperature and the nature of compounds occurring in water;

$$\mathbf{Cl_2 + P \rightarrow Cl^- + P\text{-}Cl}$$

in which:

Cl_2 – free chlorine;

P – compounds reacting with chlorine;

Cl^- – chlorides;

P – Cl - compounds resulted from oxidation with chlorine

The reaction velocity for chlorine consumption may be written:

$$\mathbf{v = \frac{d(Cl_2)}{dt} = k_{Cl_2P}\left[Cl_2\right]\left[P\right]} \quad (1)$$

The reaction velocity depends on:

- the kinetics constant which depends on water temperature; the compounds which react with chlorine; the chlorine nature (HClO or ClO_2);

- chlorine concentration;

- concentration of reaction product.

Integrating the equation (1) we obtain:

$$\mathbf{\left[Cl_2\right]_t = \left[Cl_2\right]_0 \exp(-k\left[P\right]t)} \quad (2)$$

or

$$\mathbf{\left[Cl_2\right]_t = \left[Cl_2\right]_0 e^{-k[P]t}} \quad (2')$$

Since the concentration of the compound that reacts with $\mathbf{Cl}_2$ is generally unknown the apparent (global) kinetic constant of first order is introduced.

$$\mathbf{k' = k\left[P\right]}$$

The equation ($2'$) becomes:

$$\mathbf{\left[Cl_2\right]_t = \left[Cl_2\right]_0 \exp(-k't)} \quad (3)$$

The k' is determined by representing logarithm in e base of Cl_2 concentration in function of time (that is a straight-line). The straight slope is equal with k'. Two determinations are necessary at a certain period of time (the initial and final moment)

Equation (3) will be used for simulating concentration in free Cl_2 depending on time.

This model is considered in (Gagnon, 1997; Smith et al., 1997), as well.

4. Hypothesis for modeling Cl_2 concentration in the water distribution network

Since in equation (3) a variable is the time of water circulation in network, at first should be done the hydraulic calculus of the distribution network for established the flow velocity an the network pipelines and implicitly the moving time between nodes (t = l (the pipeline length) / v (water velocity).

The hydraulic calculus can be done in steady-state (pre-established values for water consumption) or dynamic (for an extended period of simulation). For this paper the steady state is considered.

For the qualitative aspects it is considered:

- the flow is of piston type, without an axial dispersion (dispersion may be taken into account by an advective term corresponding to Wu et al. (1997) that complicates the calculus a great deal; the practical results being modified insignificantly).

- at nodes the mixing is done perfectly;

- the chlorine migration in the network respects the mass conservation law;

- for every pipeline, the Cl_2 concentration at the downstream end is expressed in accordance with the concentration in the upstream and after the eq. (3).

- for every node, the chlorine concentration results from the weighted mean through the flow rate of the concentration from the incident pipelines.

5. The determination of the kinetic constant k.

Kinetic constant k is determined by laboratory test using bottle test procedure (Walski ,T., et al.,2003).

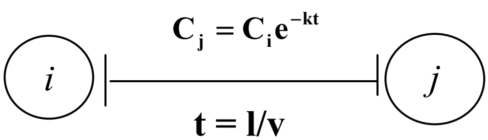
The same as for k absolute roughness coefficient and for k kinetic constant there can be introduced distinct values for each pipeline, imposed values or established values by determinations.

For the example in the paper we shall consider a global value k = 0.375 1/h.

6. The modeling of chlorine concentration in the distribution network.

For every pipeline the evolution of chlorine concentration is done with the equation (3) (fig.1)

Fig.1



For every node (fig.2.) the chlorine concentration is calculated with relation (4):

$$\mathbf{C_j = \frac{C_{j1}Q_1 + C_{j2}Q_2 + ...}{Q_1 + Q_2 + ...}} \qquad (4)$$

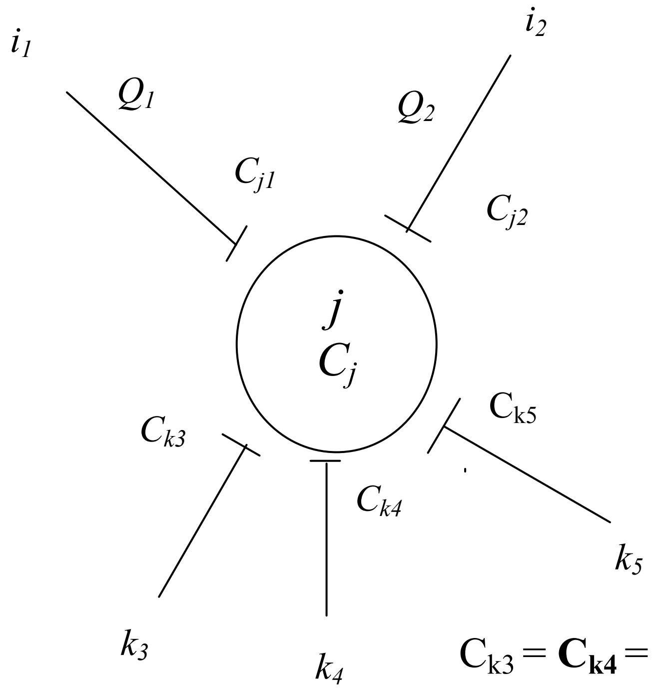


Fig. 2

$C_{k3} = \mathbf{C_{k4}} = C_{k5} = C_j$

For a pipeline there are two unknowns (initial C, final C) and they may be written T equations (3) (T – number of pipelines).

For nodes there may be written N equations (N –number of nodes) (4), in every node being an unknown

It results, apparently, that the number of unknowns is 2T + N but the number of equations that may be written are de T + N .in number.

In fact according to fig.3, the initial chlorine concentrations of the pipelines are always known, so that the chlorine concentration at nodes and the ends of the pipelines results from the solving of a T + N system of equations with T + N unknown

To generate the T + N system of equations there must be known the elements for defining the functioning hydraulic regime of the network and the kinetic constant k for the entire network or for every pipeline. The calculus algorithm comprises two steps:

Step 1. The hydraulic calculus effectuation after Barsan and Ignat (1998) or other programs. Water velocity determination in the network pipelines.

Step 2. With the relations (3) and (4) and k experimentally determined or imposed it is calculated the free chlorine concentration in mg/dm^3 for nodes and the ends of the pipelines.

7. Example

It is considered the network of town B made up of 167 nodes, 218 pipelines, 2 sources of water supply that bring water in nodes 1 (Q_1 = 2.112 l/s) şi 2 (Q_2 = 642 l/s) (fig.3).

It is considered that in nodes 1 and 2 a dose of chlorine of 0.5 mg/dm^3 is introduced. A general value is imposed for reaction constant k = 0.375 1/h.

The hydraulic calculus of the network is done after Barsan and Ignat (1998).

The calculation of the free chlorine concentration is done at the ends of the pipelines and at nodes. A picture of the free chlorine concentration evolution in the network is presented in fig. 3. It is found that in nodes 158 up to 162 the free chlorine concentration is comprised between 0.1…0.2 mg/dm^3, under the accepted limit of Romanian Law 458-2002

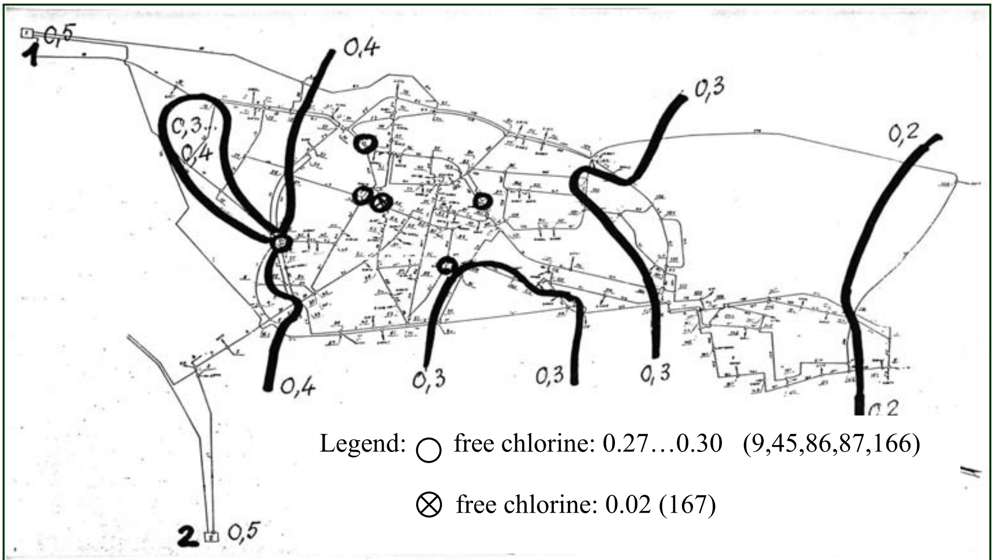


Fig.3. Free chlorine evolution in network (mg/dm^3)

In the concentration zone 0.3…0.4 mg/dm^3 five nodes (9, 45, 86, 87, 166) have a concentration comprised between 0.27…0.30 mg/dm^3 and one (167) has the free chlorine concentration of 0.02 mg/dm^3 very little in comparison with the required limit of Law 458-2002 (0.25 mg/dm^3). In the remain nodes the chlorine concentration is comprised between 0.2…0.5 mg/dm^3.

8. Conclusions

1. For the chlorine concentration modeling in the distribution networks a calculus is made comprising two steps:

a – The hydraulic calculation of the distribution network with the calibration of the model on the concrete analyzing system for establishing flow velocity in the network pipelines

b – The calculation of chlorine concentration at nodes and the ends of the pipelines achieved with a linear algebraic system of T + N equations knowing or imposing the kinetic constant value for the chlorine reaction

2. The modeling of a water distribution network quantitatively and qualitatively has a practical value if there are correctly appreciated the pipeline k coefficients, that is k the absolute roughness of the network pipeline walls for a hydraulic analysis and k the kinetic constant of the chlorine reaction (global, at sectors or at each pipeline) for modeling the chlorine concentration at nodes and the network pipelines .

3. The calibration of the quantitative numeric model (hydraulic) and qualitatively (chlorine concentration) represents an up to date problem in water distribution in localities.

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OPTIMIZATION OF CHLORINE DOSING IN DRINKING -WATER STORAGE TANKS

Emanoil BÂRSAN[1], Călin IGNAT[2]

Abstract

The quality of the stored water in the drinking water tanks is considered to be proper if:

1 – the residence time in storage tanks is under the limit stipulated by standards;

2 – the mixing of the existing water with that entered afterwards is complete done;

3 - chlorine, with the disinfectant role (the primary disinfection) introduced in the tank assures after the reactions in the tank, the required chlorine in the distribution network.

In the case of an inflow-outflow tank with a single compartment, working with a steady flow, where there is admitted that the disinfection reaction is of order one in comparison with its concentration, and the mixing is complete, the specialized literature offers the relations for following the link between the chlorine concentration at outflow (C_{out}) and inflow (C_{in}) (checked experimentally):

In the case of an inflow-outflow tank with a variable inflow and outflow, where there are the problems of integration, it is suggested, by authors, an iterative procedure based of the existent relations of anterior case.

In addition, for the chlorine dose introduced in the supply pipe it is established a dosing program, realized by a simplex procedure, for the minimizing of chlorine consumption in 24 hours, so that at the outlet from the tank to be assured, hour by hour, the required dose for the necessary secondary disinfection of the water distribution network.

In the end, it is presented an application of the proposed procedure at a tank for which the fluctuant volume develops between the emergency and total capacity of the tank.

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1. Introduction

Water storage reservoirs are used in drinking water distribution systems for the compensation between supply and consumption, the assuring intangible reserve for fire/failure and the pressure for the case of distribution systems working gravitationally.

Due to large water storage volumes in these structures there are recorded incomplete mixings of existent water with new introduced water quantities, the fact which conduct to water quality deterioration for passed water through these constructions.

At the same time the storage tanks are used for chlorine introducing for the disinfection goal, so as at water passing to distribution network will be assured required bacteriostatic chlorine quantities of Law 458/2002 (0.5 mg/dm^3).

In ensemble the tank may be considered as a reactor in which the introduced chlorine through supply conduit acts through oxidizing capacity on organic substances and microorganisms that have passed of treatment stage

The corresponding quality of the stored water in the drinking water tanks is considered to be proper if (Rossman and Grayman, 1999):

1 – the residence time in storage tanks is under the limit stipulated by standards;

2 – the mixing of the existing water with that entered afterwards is complete done;

3 - chlorine, with the disinfectant role (the primary disinfection) introduces in the tank assures after the reactions in the tank, the required chlorine in the distribution network suited to Law above mentioned.

4 - thermal stratification do not appears due to temperature difference between the new introduced water and water existing in tank

In usage there are two types of tanks having in view their working way:

1. flow-through tanks, in which it takes place simultaneously both the supply and consumption;

2. inflow/outflow tanks, in which it takes place successively the filling (when the consumption in network is less than the supply flow) and emptying (when the consumption in the network is greater than the supply flow of network) .

For modeling disinfectant residual in tanks there are applied the model of compartments (Rossman, 2000; Clark et al., 1996; Mau et al, 1995; Rossman et al., 1995) – a model for the system of black box type in which the hydrodynamic processes which takes place there are represented only under a conceptual form by a number of compartments which simulates the zones of normal (perfect) mixing and short circuiting or the dead zones by tank volume discreteness

In the frame of compartment it is considered that the mixing is perfect and if the substance is consumable (e.g. chlorine) takes place a consumption reaction of first order

The link between compartments and the outer (supply, distribution) is done by the continuity relations.

In order to realize / built-in and utilize a model there are made hypothesis that concern the inflow and outflow (constant or variable), the flow between compartments (Clark et al.,1996; Rossman, et al.,1995), the flow direction (one way flow or bi-dimensional), the type of substance having been in view (with or without reaction).In function of degree of accuracy the models may with one, two or three compartments.

2. Modeling disinfectant residual for inflow-outflow tanks with one compartment (Rossman, L., A. et al., 1995).

For the modeling of chlorine consumption process are accepted two hypothesis:

1 – the tank content is complete mixing. The conditions for this hypothesis it is presented and studied by Rossman and Grayman (1999).

2 – the disinfectant reaction is of first order in relation with the disinfectant concentration. For disinfectant consumption (Cl_2) it is applied a first order reaction of the form:

$$\mathbf{C_{out} = C_{in}\exp(-kt)} \tag{1}$$

The coefficient k is established by laboratory tests on the samples of tank water.

In this case the operation is done in two steps (first the filling, then the emptying). The disinfectant (chlorine) is introduced in reservoir in filling stage, it reacts with oxidizable substances from water mass and it leaves to distribution network in the emptying stage.

With the above mentioned hypothesis, the differential equations which characterize the water and disinfectant mass conservation of reservoir on the filling period are:

$$\mathbf{\frac{dV}{dt} = Q_{in}} \tag{2}$$

$$\mathbf{\frac{d(VC)}{dt} = Q_{in}C_{in} - kVC} \tag{3}$$

and during the emptying period they are:

$$\frac{\mathbf{dV}}{\mathbf{dt}} = -\,\mathbf{Q}_{\mathbf{out}} \tag{4}$$

$$\frac{\mathbf{d(VC)}}{\mathbf{dt}} = -\mathbf{Q}_{\mathbf{out}}\mathbf{C} - \mathbf{kVC} \tag{5}$$

where V – volume of water in the tank; C – concentration of disinfectant in the tank; Q_{in} – flow rate into the tank; Q_{out} - flow rate out of the tank; k – reaction rate constant; C_{in}- the disinfectant concentration of inflow rate. The establishment of disinfectant concentration in the tank is done for the case when the tank is operated with a inflow rate (Q_{in}) and outflow rate (Q_{out}) constant, a hypothesis which may be admitted during the intervals of a hours during one day.

Under these conditions the volume of storage will fluctuate between V_{out} at the start of the fill cycle and V_{out} + V_a at the end of the fill cycle in which V_{out} represents the static (intangible) reserve volume necessary for emergency purposes (e.g. fire - fighting); V_a is the compensation volume at the consumption demands.

At the start of each emptying C = C_{in} (the disinfectant concentration at the end of the anterior reservoir filling). At time t_{out} at the end of the emptying, the disinfectant concentration C_{out} (the minimum concentration with which enters the distribution network resulting from the solving of the equations (4) and (5) are:

$$\mathbf{C}_{\mathbf{out}} = \mathbf{C}_{\mathbf{in}}\,\mathbf{exp(-kt_g)} \tag{6}$$

Solving (4) (with V = V_{out} for t = 0) it is obtained:

$$\mathbf{V} = \mathbf{V}_{\mathbf{out}} + \mathbf{Q}_{\mathbf{in}}\mathbf{t} \tag{7}$$

For solving of the equation (5) and the using for V of the relation (7) it is obtained differential equation for the disinfectant concentration during the fill cycle:

$$\frac{\mathbf{dC}}{\mathbf{dt}} = \frac{\mathbf{Q}_{\mathbf{in}}(\mathbf{C}_{\mathbf{in}} - \mathbf{C})}{\mathbf{Q}_{\mathbf{out}} + \mathbf{Q}_{\mathbf{in}}\mathbf{t}} - \mathbf{kC} \tag{8}$$

The solution to equation (8) with the initial condition C = C_{in} at t = 0, give the following equation for C_{in} at time t_{in} at the end of the fill cycle:

$$\mathbf{C_{out}} = \frac{\mathbf{C_{out}\theta \exp(-kt_{in})}}{\mathbf{(\theta+t_{in})}} + \frac{\mathbf{C_{in}\left[1-\exp\left(-kt_{in}\right)\right]}}{\mathbf{k\left(\theta+t_{in}\right)}} \tag{9}$$

where $\theta = \frac{\mathbf{V_{out}}}{\mathbf{V_{in}}}$. Substituting C_{in} from expression (9) in (4), it is obtained the disinfectant concentration at the departure from tank C_{out} relate to C_{in} introduced during the fill period :

$$\frac{\mathbf{C_{out}}}{\mathbf{C_{in}}} = \frac{\mathbf{\exp\left(-kt_{out}\right)-\exp\left[-k\left(t_{in}+t_{out}\right)\right]}}{\mathbf{k\left(\theta+t_{in}\right)-k\theta\exp\left[-k\left(t_{in}+t_{out}\right)\right]}} \tag{10}$$

It is define the three dimensionless variables in function of operation parameters of the tanks:

$$\mathbf{T = k(V_{out} + V_a)/Q_{in} \;;\; S = V_{out}(V_{out} + V_a) \;;\; F = V_{in} / V_{out} = t_{out} / t_{in}}$$

where : T – the product of the reaction constant and the refill time for an empty tank; S is the ratio of emergency storage to total tank volume; F – the ratio of the filling flow rate to the emptying flow rate or, equivalently, the emptying time to the filling time.

With these expressions, the relation (10) becomes:

$$\frac{\mathbf{C_{out}}}{\mathbf{C_{in}}} = \frac{\mathbf{\{1-\exp\left[-T(1-S)\right]\}\exp\left[-T(1-S)F\right]}}{\mathbf{T\{1-S\exp\left[-T(1-S)(1+F)\right]\}}} \tag{11}$$

The validity of application of this relation for the type of the studied tank it is confirmed by the experiments performed on the three reservoirs from USA, at natural – full (Rossman et al., 1995)

3. Optimization of chlorine dosing for a inflow-outflow tank

For the case of a flow - through reservoir modeled as a compartment with the perfect mixing and the concomitant with the supply and drainage – distribution, the pursuit of chlorine concentration evolution may be realized by the two ways:

- solving the differential equation (1) from Mau et al. (1995) specific to the flow-through reservoirs, but with the problems at integration specified in (Mau et al.,1995) and the necessity to appeal to the simplifications, or, the authors proposal;

- using of the equation (11) specified to the inflow/outflow reservoirs the minute after the minute for each hours of the day, this is, the established minute by minute for the each hour of the day the chlorine concentration as pursuit of the mixing of water volumes and a chlorine consumption as oxidant , in two successive steps (filling with inflow rate Q_{int} and emptying with outflow specified to respective minute).

Between the two way of the chlorine concentration establishing is has been found that the differences are insignificant

In addition, for the chlorine dose that is introduced in the supply pipe it is established with a dosing program realized by a simplex procedure of minimizing chlorine consumption in 24 hours, so that at the outlet from the tank to be assured, hour by hour the required dose for the secondary disinfection specified by Law 458-2002.

The application of proposed procedure it is realized for the system presented in (Barsan and Ignat, 1998), where in the node 2 it is considered a reservoir of 10,000 m^3. It is imposed as the fluctuant volume of reservoir to evaluate between 3,000 m^3 (intangible reserve) and 10,000 m^3 (maximum capacity of the reservoir) In table 1 it is presented, hour by hour the evolution of water volume, inflow rate and outflow rate and the optimized chlorine dose that should be introduced in supply conduit so that in the output conduit should be the chlorine dose required (0,5 mg/dm^3) (it is considered $k = 1$ day^{-1})

Table 1.Evolution of chlorine concentration in a storage reservoir of 10,000 m^3. Initial volume 4500 m^3 (hour 24). The minimum concentration prescribed at the outlet: 0.5 g/dm^3.The inlet concentrations (C-Input) have been determined by the minimization of chlorine consumption

Hour	Volume (m^3)	Inputs (l/s)	C-Input (mg/l)	Outputs (l/s)	C-out (mg/l)
1	5619,168	642	0.550	331,12	0.50
2	6738,336	642	0.560	331,12	0.50
3	7857,504	642	0.570	331,12	0.50
4	8976,672	642	0.580	331,12	0.50
5	9541,152	642	0.590	485,20	0.50
6	9550,944	642	0.595	639,28	0.50
7	9133,344	642	0.595	758,02	0.50
8	8564,544	642	0.591	800,00	0.50
9	7693,104	642	0.586	884,06	0.50

10	6897,504	642	0.579	863,00	0.50
11	6101,904	642	0.571	863,00	0.50
12	5306,304	642	0.564	863,00	0.50
13	5204,064	642	0.557	670,40	0.50
14	5101,824	642	0.556	670,40	0.50
15	4722,240	642	0.556	747,44	0.50
16	4065,312	642	0.552	824,48	0.50
17	3408,384	642	0.546	824,48	0.50
18	3028,800	642	0.541	747,44	0.50
19	3000,000	642	0.537	650,00	0.50
20	3007,200	642	0.537	640,00	0.50
21	3014,400	642	0.537	640,00	0.50
22	3000,000	642	0.537	646,00	0.50
23	3331,200	642	0.537	550,00	0.50
24	4500,000	642	0.540	317,33	0.50

The input concentrations may be utilized by a operator as the program of chlorine dosing.

4. Conclusions

1. Water storage reservoirs are used in drinking water distribution systems for the compensation between supply and consumption, the assuring intangible reserve for fire/failure and the pressure for the case of distribution systems working gravitationally.
2. The quality of the stored water in the drinking water tanks is considered to be proper if : a – the residence time in storage tanks is under the limit stipulated by standards.; b – the mixture of the existing water with that entered afterwards is complete done; c - chlorine, with the disinfectant role (the primary disinfection) introduced in the tank assures after the reactions in the tank, the required chlorine in the distribution network to maintaining water biostability during the transit through distribution network..
3. In the case of an inflow/outflow tank with a single compartment, working with a constant flow, where there is admitted that the disinfection reaction is of order one in comparison with its concentration, and the mixture is complete, the specialized literature offers the relations for following the link between the chlorine concentration at outflow (C_{out}) and inflow (C_{in}) (checked experimentally):
4. In the case of an inflow/outflow tank with a variable inflow and outflow, where there are the problems of integration, it is suggested, by authors, an iterative procedure based of the existent relations of anterior case.

5. In addition, for the chlorine doze that is introduced in the supply pipe it is established a program for chlorine dosing in the inlet pipe of tanks, by a simplex procedure that minimize the chlorine consume in 24 hours, and assures at the outlet pipe of tanks, every hour of day, the required dose of 0.5 mg/dm^3, the program which may be inserted in a simulator of water quality of the system.

6. In the end, it is presented an application of the proposed procedure at a tank of 10,000 m^3 for which the fluctuant volume develops between the emergency and total capacity of the tank.

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OPTIMIZATION OF LAMELLA SETTLERS

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Abstract

Lamella settlers are installations used for water clarifying. There are made up of n parallel plates (tubes) system, inclined with an α angle from the horizontal through which water containing suspensions is clarified by a process of liquid – solid separation through gravitational way and in laminar conditions.

Starting from the current forms met in practice for lamella settlers, both the independent and combined with other solid-liquid separation installation (superpulsator and pusatube clarifiers) it is studied their insertion into cyndrical or parallelipipedic tanks

For the optimal design of lamella settlers inserted into cylindrical or paralelipipedic tanks it is adopted as criterion of optimization the minimization of the critical settling velocity depending on the geometrical elements of installation and the entering velocity in the tank

In function of inclination angle of lamella module, it is established maximum efficiency and geometrical elements of installation for some significant cases

For the optimization of the lamella settlers is realized a simulation of behavior of this installation for a certain domain for the parameters involved in designing. From the multitude of efficient function values is searched minimum value establishes the best relative value of size which may be retain at given upward velocity.

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1. Introduction

Lamella settlers are installations used for water clarifying. There are made up of n parallel plates (tubes) system, inclined with an α angle from the horizontal through which water containing suspensions flows in laminar condition.

Lamella settlers have appeared after being found that superficial loading does not depend of the sedimentation tank depth.

In this installation the particle with w settling velocity greater than w_c, critical settling velocity separate themselves from the fluid and they set down. The removed suspensions slide on the inclined planes formed of parallel plates of the installation, they are collected and then discharged.

The major advantage of this installations is continuous working with high efficiency in comparison with classical installation of sedimentation.

For analyzing the following assumptions are made: a – water flowing is laminar and one dimensional; b – the initial acceleration of the particles up to terminal settling velocity is neglected; c – in installation, the process of coagulation – flocculation of the particles does not continue; d – the terminal settling velocity of particle does not change with time or position; e – for the water velocity between the plates of installation it is taken into account the average value v_d (fig. 1)

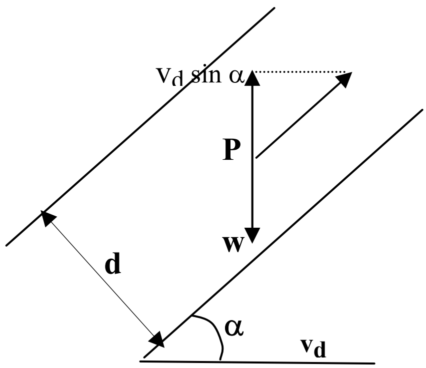

Fig. 1

The displacement of a particle P carried away by the water stream is achieved by common action of flow and settling velocity (fig. 1).

According to AWWA (1999) for lamella settling working in countercurrent, the time t for a particle to settle the vertical distance between two parallel inclined surface is (fig. 1):

$$\mathbf{t} = \frac{\mathbf{d}}{\mathbf{v_s} \cos \alpha} \text{ with } \mathbf{v_s} = \mathbf{w} - \mathbf{v_d} \sin \alpha \qquad (1)$$

where d is the perpendicular distance between surface and α is the angle of surface inclination from the horizontal; v_d – velocity between the surfaces.

The length of surface L_p needed to provide this time, if the liquid velocity between the surfaces is v_d, is:

$$\mathbf{L_p = \frac{d(w - v_d \sin\alpha)}{v_s \cos\alpha}} \tag{2}$$

By rearranging this equation, there is obtained following value for v_s .

$$\mathbf{v_s = \frac{v_d}{\sin\alpha + \frac{L_p}{d}\cos\alpha}} \tag{3}$$

A system of lamella tanks will be more efficient the more it retains particles with smaller dimensions, that is, they have a slower settling velocity v_s.

For an average flow velocity, v_d, and given type of system (plates, tubes, etc.) the maximum efficiency is obtained by minimizing of expression (4):

$$\mathbf{E = \frac{v_s}{v_d} = \frac{1}{\sin\alpha + \frac{L_p}{d}\cos\alpha}} \tag{4}$$

Starting from the fact that in practice, both the independent and combined installation, the lamella systems are inserted into cylindrical or parallelipipedic tanks, it is necessary a corresponding adjustment of relation (4) and its study for the characteristic cases.

2. Lamella Settler Inserted into a Vertical Cylindrical Tank with Upflow (fig.2a)

The relation (4) establishes the efficiency for the general case of lamella tank. When the lamella settling installation is inserted into a cylindrical tank, the average flow velocity depends on the rectangular section inserted into cylinder circle, the angle of inclination and the plate length of installation.

Considering the geometrical elements from fig.2 a it may written successively:

$$\frac{v_b}{v_0} = \frac{\pi R^2}{a(4R^2 - a^2)^{1/2}} \tag{5}$$

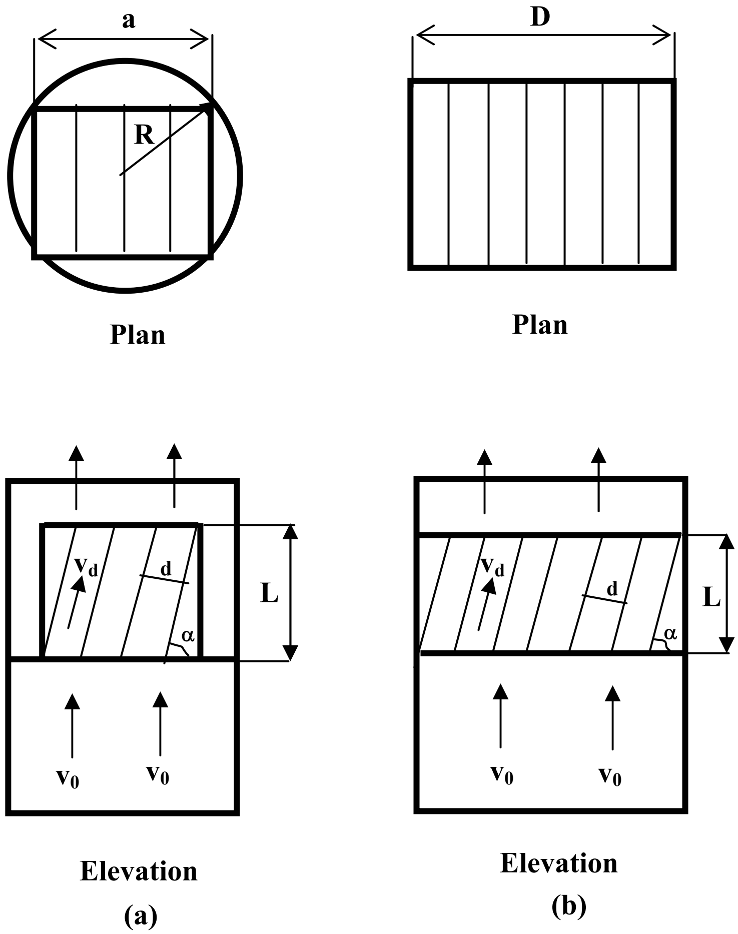

Fig. 2

$$\frac{v_c}{v_b} = \frac{a}{a - L\,\mathrm{ctg}\alpha} \tag{6}$$

$$v_d = \frac{v_c}{\sin\alpha} \tag{7}$$

Combining the three relation, it results:

$$v_d = \frac{\pi v_0}{\left[4-\left(\frac{a}{R}\right)^2\right]^{1/2}\left(\frac{a}{R}-\frac{L}{R}\mathrm{ctg}\alpha\right)\sin\alpha} \tag{8}$$

Introducing relation (8) in relation (4), it results:

$$E_1 = \frac{w_c}{v_0} = \frac{\pi\delta}{\left(4-\beta^2\right)^{1/2}\left(\beta-\gamma\ \mathrm{ctg}\alpha\right)\left(\gamma\cos\alpha+\delta\sin^2\alpha\right)} \tag{9}$$

with:

$$\beta = \frac{a}{R}; \qquad \delta = \frac{d}{R}; \qquad \gamma = \frac{L}{R}$$

3. Lamella Settler Inserted into a Vertical Parallelipipedic Tank with Upflow (fig.2b).

In this case relations (5)…(8) become:

$$v_b = v_0 \tag{5'}$$

$$\frac{v_c}{v_b} = \frac{D}{S - L\ \mathrm{ctg}\ \alpha} \tag{6'}$$

$$v_d = \frac{v_c}{\sin\alpha} \tag{7'}$$

$$v_d = \frac{v_0}{\left(1-\frac{L}{D}\mathrm{ctg}\ \alpha\right)\sin\alpha} \tag{8'}$$

By introducing of relation (8’) in relation (4), the efficiency has the following expression:

$$E_2 = \frac{\delta}{\left(\sin\alpha-\gamma\cos\alpha\right)\left(\gamma\ \mathrm{ctg}\alpha+\delta\sin\alpha\right)} \tag{10}$$

with:

$$\delta = \frac{\mathbf{d}}{\mathbf{D}} \qquad ; \qquad \gamma = \frac{\mathbf{L}}{\mathbf{D}}$$

4. The determination of Maximum Efficiency

The maximum efficiency for those two cases taking into consideration is established by searching the minimum of expressions E

Because in E expressions there are many parameters, in this paper, α (the inclination angle of lamella) was considerated as basic parameter, that is, $E = f(\alpha)$. Then it was calculated $\mathbf{f}'(\alpha)$. It was solved the equation $\mathbf{f}'(\alpha) = \mathbf{0}$, by the Newton method for the specified parameters (in expressions E_1 - β, δ, γ and in expressions E_2 - δ, γ). The zero obtained is the minimum point for functions E.

For a more comprising analyze there were calculated the minimum values of E for different values of δ (δ = 0,02, 0,05, 0,1, 0,15, 0,2 m) and angles α comprised between 5^0 …85^0 (steps of 5^0) resulting the corresponding values for the other parameters.

In figs 3 and 4 functions E1… E2 depending on angle α and for the pre-established value of δ (distance between lamellae) are represented (as examples)

With these diagrams and the associated program there can be dimensioned optimally the lamella moduli inserted in cylindrical and parallelipipedic tanks.

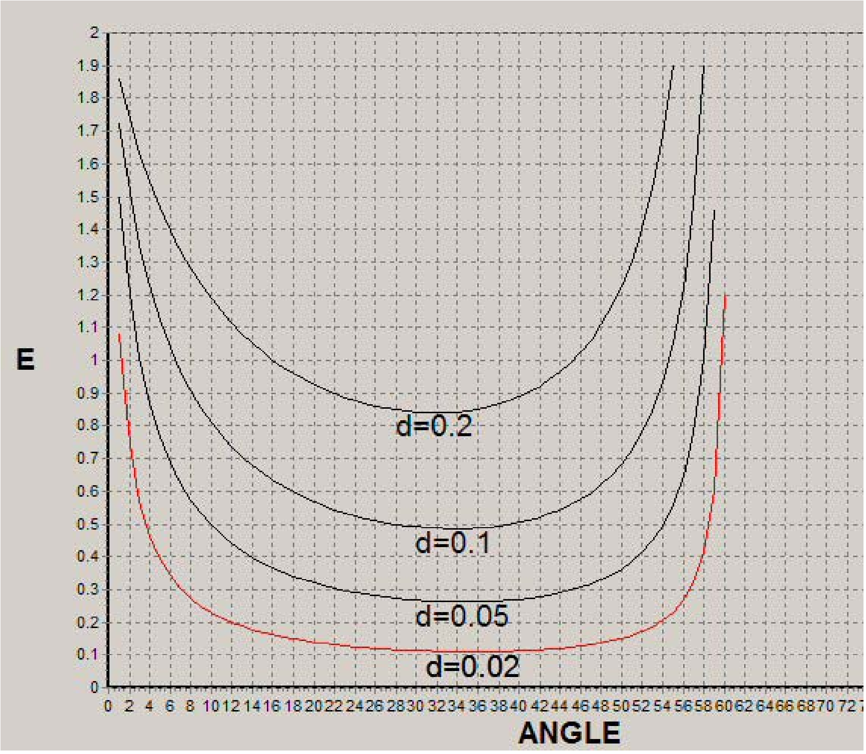

Figure 3

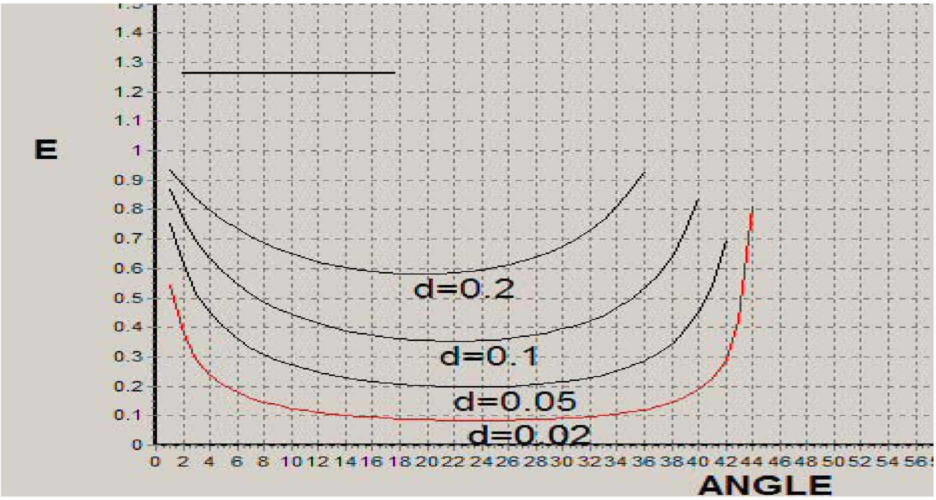

Figure 4

5. Conclusions

1. For the optimal design of lamella settlers inserted into cylindrical or parallelipipedic tanks it is adopted as criterion of optimization the minimization of the critical settling velocity (v_{vs}) depending on geometrical elements of installation

2. The lamella settler has the greater efficiency, the smaller angle of inclination is, and the smaller distance between the plates is more reduced.

3. For the lamella settlers inserted into cylindrical tanks and the studied cases a $\approx$ 1,6 R (fig.2.a)

4. For establishing geometrical elements and the maximum efficiency at different inclination of the lamella settler we use diagrams of types presented in figs.3 and 4.

5. Because there must be achieved a gravitational moving off of the sludge from installation the inclination angle of the plates is of $50^0 \ldots 60^0$, fact that moves off the installation from the minimum efficiency.

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CONSIDERATIONS ON INTEGRATED BRIDGE DESIGN SYSTEM IN CYBERNETIC CONCEPT

Constantin IONESCU[1]

Abstract

The paper presents some consideration concerning the construction of an integrated bridge design system based on three fundamentals concepts adopted from systems engineering, bridge convergent engineering and informatics.

Systems engineering introduced the concept of total realization of the bridge, corresponding to the four stages the bridge passes through its existence: design, realization, operational stage, and technical-economical evaluation phase. All four compose a closed system with the outputs influencing the inputs.

Convergent engineering of the bridges underlines the design concept by integration of all phenomena emerging on the life cycle of a bridge, due to external factors and resistance structure definition parameters.

Informatics intervenes in integrated bridge design through the techniques of monitoring the phenomena to obtaining data, information, and knowledge, and through effective use of computers in the design process.

The papers develops around a logical diagram, which comprises 22 phases concerning the integrated bridge design system. Among them we presented extensively: study of social demand, estimations including pre-feasibility and feasibility, determination of the environmental module, identification of the universe of the bridge, defining the qualities of the processes from the bridge – system, design processes characteristics and structural characteristics.

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1. Introduction. Systems engineering. Convergent engineering.

The structural designer is an engineer open-minded for the new, possessing profound technical and scientific knowledge. Any novelty emerging into a science that interacts with the structural engineering must be immediately analyzed and, in most cases, included in the design of new structures, of new bridges.

The systems theory and informatics, with its spectacular and effective progress, have opened new ways that, when correct learned, may contribute to the emergence of new types of structures or to arising the technical level of the existing ones trough modernization and optimization of the design process.

Thus, systems engineering imposed the concept of total bridge design. According to this concept, the four phases the bridge passes trough along its existence: design, realization (construction of the bridge), operation (experiments, maintenance etc.), and evaluation (costs, performance, reliability, esthetics etc.). The four phases form a closed system with feedback, the output influencing the inputs.

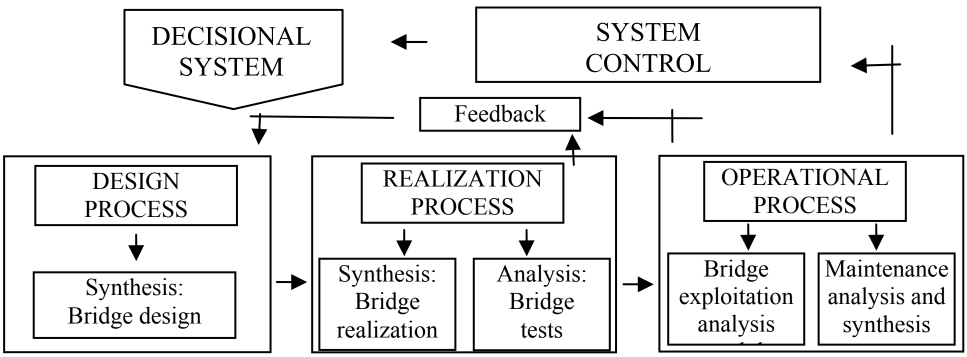

Fig.1. Bridge engineering cycles

Also, the convergent engineering, as it is presented in the mecatronic systems theory, represents a design methodology that incorporates all phenomena occurring over the life cycle of the system (i.e. the bridge) considering all its quality and safety characteristics, thus its global effectiveness. Convergent engineering represents a fundamental strategy based on use of computers and oriented to three main objectives: reduction of the time between conception and realization, and increase of the quality of the processes.

2. Integrated design system in cybernetic concept

Fig. 2 presents in a logical diagram the main stages of this integrated concept. Following, each important stage of this system is analyzed. Follow-up of the explanations is eased by the fact that each nominated stage is accompanied by a number that appears later in the analysis. We also addressed adjacent topics not included in the figure but necessary to the explanation.

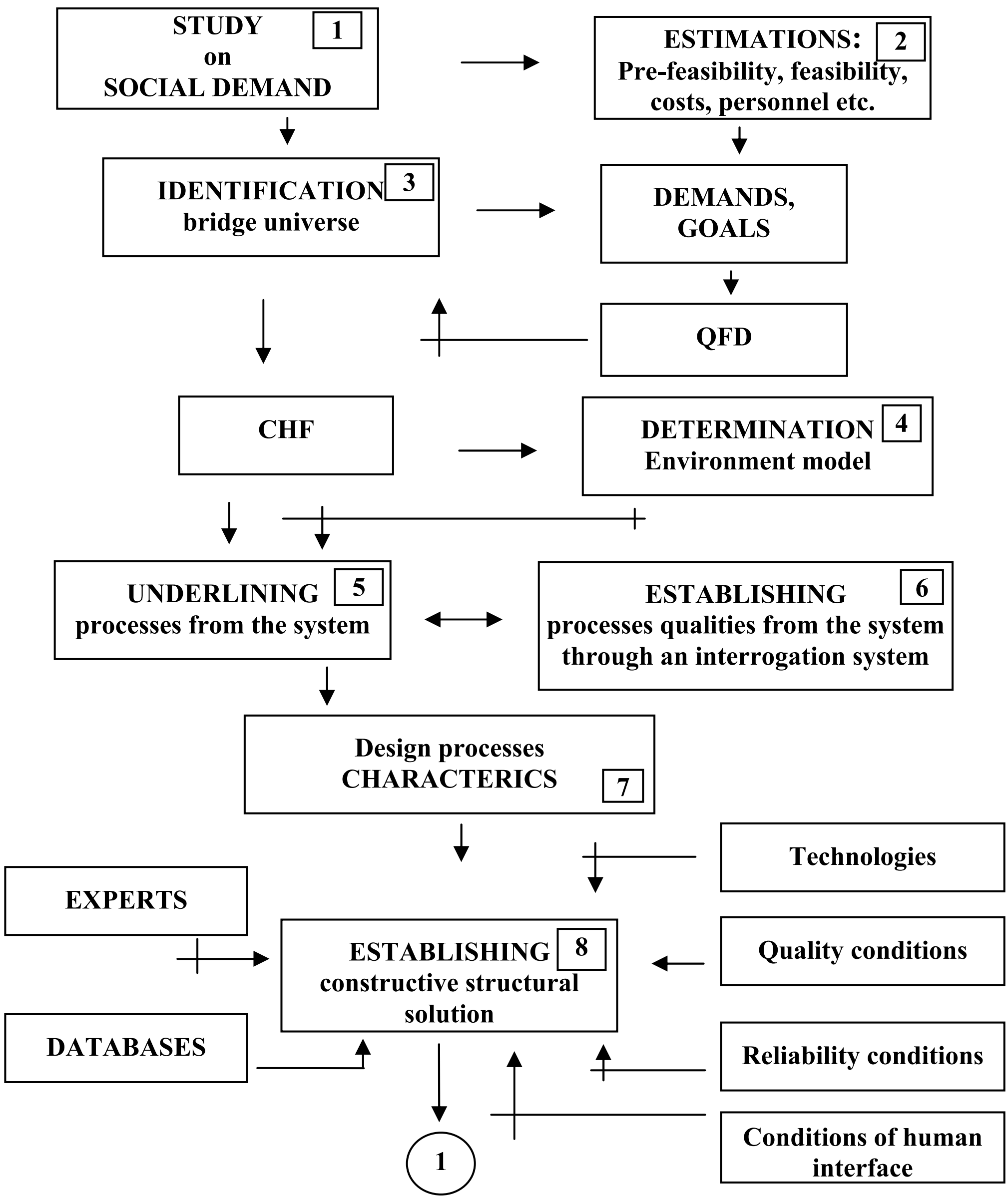

Fig.2. Integrated system of bridge design in cybernetic concepts

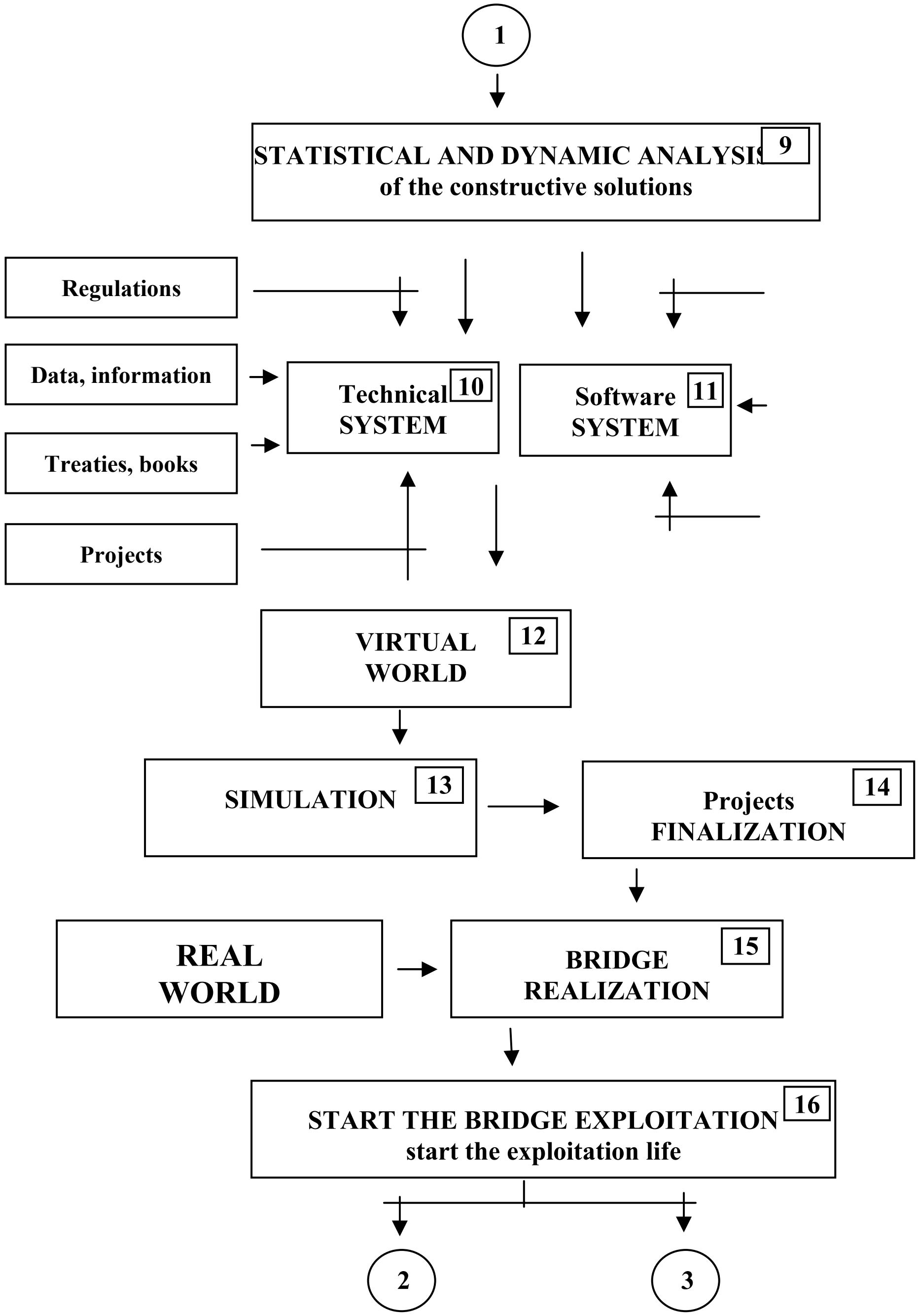

Fig.2. (cont.). Integrated system of bridge design in cybernetic concepts

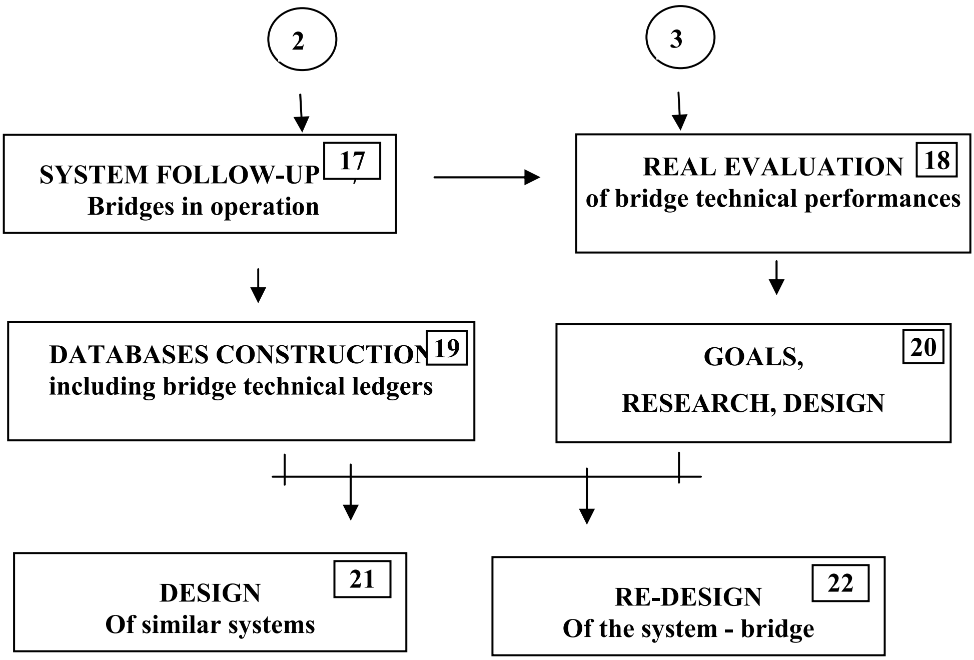

Fig. 2. (cont.). Integrated system of bridge design in cybernetic concepts

2.1. Study of social demands

The social demand transmited to a design team incloses the desing theme, required level of performance, and the resouces available for the realization of the bridge. Analysis process of the social demand, initialized immediately after receiving the demand, may concequatly lead to change in the demand, with the agreement of the beneficiary, to improve the performences according to the design principles, but in the same time the design principles may modify to reflect the prescribed social demand.

2.2. Estimations: pre-feasibility, feasibility, costs, personnel

In this stage the project manager fulfills the pre-feasibility and feasibility studies, which represents the first two phases in the conception of the bridge design. Trough these two phases one realizes evaluations on the project itself concerning total value, costs for feasibility studies and getting required approvals, setting main characteristics of the bridge, including description of the solution, main technical-economical indicators etc.

2.2.1 Demands. Goals.

After resolving the stage on the estimations of social demand, the design engineer will highlight the general objectives for the project and the goals for each sub-project.

Parallel with valuation of the objectives of the stages of the design process, the requirements on the project are decided especially referring to technical system, software system, and human factor (CHF). The requirements are of technical, quality, and reliability nature.

2.3 Identification of bridge universe

This stage represents a specific module of the convergent engineering integrated into design concept of complex systems like the bridge. From abstract point of view, the universe of the bridges is an infinite dimensional space of the design problems and solutions and refers to:

a) Defining the domain of the universe of the problem;

b) Accounting the functions defined on the design domain;

c) Development of a function of cost through economic analysis of the problem;

d) Identification of problem's restrictions due to external factors: social demand, standards, regulations, laws etc.;

e) Minimization of the cost function which was developed in problem's analysis phase.

The universe of the bridges refers to:

a) Examination of the social request;

b) Transformation of the request into a set of objectives (sub-projects);

c) Analysis of the functions of the system to identify weak points;

d) Definition of the goals of the project;

e) Analysis of the environment.

The proposed procedure refers to all stages of the bridge life. It considers the simultaneity of bridge engineering cycles: design, realization, operation, and maintenance. This method must be enlarged through development of Quality Function Deployment – QFD – system that unifies the requirements, objectives and analyzes the competitiveness of the solutions proposed by the design process.

To define the purposes of and relations between the global objectives one uses the objectives tree method that comprises:

a) Expansion of the global list of objectives;

b) Ordering the list of the set of objectives grouped on hierarchical levels: inferior, superior etc.;

c) Construction of a tree (diagram) of the objectives including indication (identification) of the hierarchical relations and inter-connections; resulted braches represent the relations suggesting the means to achieve the objectives.

This activity is continued with the image of a detailed model of functions, corresponding to the ordered set of objectives. The method applied in this case relies on the structural analysis which underlines two models:

a) Environment model, where the complex system, which is being designed, is situated in the context of the real world and it will comprise beside human factor other systems;

b) Essential model of systemic behavior, finding the inputs and outputs of the system - the bridge (using statistical, dynamic, seismic analysis etc.).

2.3.1. Quality Function Deployment (QFD)

Quality function QFD fundaments a method consisting in finding a system of matrices: matrix that integrates the requirements and the objectives; matrix of correlation between parameters and demands – level of realization; concurrence matrix (solutions already existing) and solution competitiveness matrix proposed through design.

2.3.2. Creative human factor (CHF)

Creative human factor plays an essential role in solving the design process. In modeling human uncertainty the following aspects must be observed::

a) human factor is a system that is asked to give real-time answers and to realize them correctly;
b) human factor has all the possibilities to belong to the design process, to the design system;
c) human factor presents special capabilities and resources to process data, to decide, be flexible, be adaptive and to answer to stress.

The interface of the system with humans, with the decision function, must consider the complementary aptitudes of the two systems (human and technical achievement). Human aptitudes might be: intuition, capacity to estimate, creativity, adaptive capacity, associative memory, non-deterministic decision, recognition of forms, acknowledge of the universe of bridges, capacity to make mistakes.

These aptitudes must be observed when dimensioning the human system for realization of a project of a bridge, the bridge system.

2.4. Environment model

Environment model is constructed following the next steps:

a) definition of a boundary of the bridge – system, boundary that separates the system from the exterior world;
b) clear delimitation of the sub-systems of the exterior world: road, foundation terrain, river, obstacle etc.;
c) construction of the context diagram representing the processes, interactions of the system with exterior world. In this stage we account for external systems that communicate with the considered system (bridge): actions, perturbations etc.;
d) association of an "event list" to the context – diagram. The event list describes the occurrence in the environment of situations to which the bridge must respond. The list represents a "communication medium" through which the designer and the user might identify and specify the relevant events from the environment. Each event from the list is modeled in context diagram by association of the informational flows for each activity.

The model of behavior defines how the system will react to the stimuli (actions, bias) or events from the environment.

2. 5 Underlining the processes from the system

Underlining the processes from the system is done according to the definition of a technical physical system. This stipulates that the designer determines the system transition from a state into another state for all the historical time of the system's life span. Mathematical models characterize description of a process. Models can be of various types: statics, dynamics, informatics, communications with the environment and with the human factors.

2. 6 Establishing system processes qualities

Concerning the interrogation system imagined for bridge – system processes quality assessment, we consider that we can use the proposed system for the mecatronics systems processes [1], adapted for the bridge particular situation.

It is obvious that the interrogation system must be part of the expert system. Following there are some questions the design engineers must answer:

a) Is the considered process necessary for realization of the required functions? knowing that the processes are divided in:
    a. dominant processes – compulsory processes in bridge design;
    b. auxiliary processes;
    c. replaceable processes – when one of the preview processes is segmented into sub-processes;

b) Does the accepted model for environment correspond to the requirements for realization of the levels of performance, quality, feasibility etc.?
c) According to the environment model, to determine the parameters which should be accounted for so that bias effects could not be assimilated or propagated into the system?
d) Has the process the information required for system technical condition assessment?
e) Is the process dimensioned to minimize the time for its completion (its event my occur sequential, parallel or combined) and to respond trough its output to the chronological requirements of the other processes its is connected with?
f) Is the process keeping its characteristics of temporal performance and productivity, allowing in the same time fulfillment of parallel functions of survey, detection, maintenance?
g) Are process operations constructed with minimum of errors?
h) Is there a failure tolerance scheme assured in case of sudden failure and interruption of the process (i.e. in operation process)?
i) Does the human factor using the bridge understand the processes in their interactions, in such a way that its decision to have a low degree of deviation from the correct decision?

2.7 Design processes characteristics

The process characteristics associated to design stage of a system – bridge are detailed in Fig. 3.

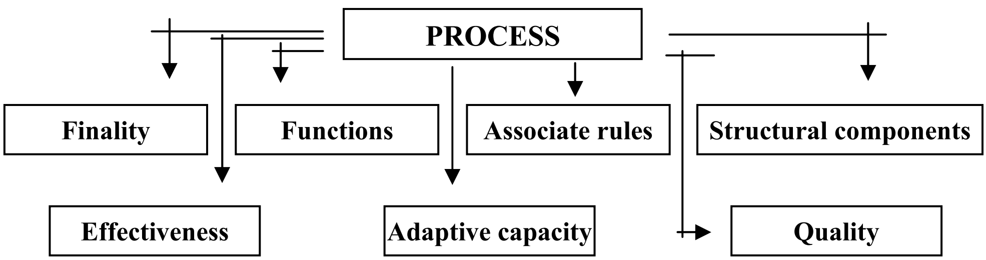

Fig.3. Process characteristics

Effectiveness represents, in this case, a measure of the productivity, the capacity of a system to get realized.

2.8 Selection of the constructive structural solution

Constructive solution for the bridge is fixed by the design engineer making appeal to experts and databases. Databases containing different bridge structures from literature: treaties, monographs, articles may be found on INTERNET or in agency's own projects on the INTRANET.

Concurrently with selection of the constructive solution it is necessary to chouse methodology of design, bridge realization technology, quality and reliability conditions and human factor condition according to its performances.

Obvious in realization of the bridge, selection of the solution is made trough pre-feasibility and feasibility.

3. Conclusions

1. The paper represents an attempt to achieve an integrated bridge design system based on three fundamental concepts adopted from systems engineering, convergent engineering and informatics.
2. This integrated design system was inspired by some similarities of the design process from bridge engineering and mecatronic systems theory.
3. The complexity of a system, the bridge, its interactions with the road, scour, foundation soil etc., imposes new design, realization and maintenance concepts which to rise the technical and reliability performances of the bridges.

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PLASTIC ANALYSIS OF CONTINUOUS THIN-WALLED BEAMS

Cezar AANICĂI [1]

Abstract

According to [3], Eurocode 3; Part 1.3, redistribution of internal forces and moments can be used for purlin dimensioning at the ultimate limit state, if the dimensioning is based on experimentally derived ***(M,θ)***-graphs. Two different behaviour types are studied: softening moment resistance and ideally plastic ***(M, θ)*** -relation, respectively. ***(M, θ)***-relation is obtained from the support detail test and design curve for ***(M, θ)***-relation is validated in the full scale test. Ultimate load yielding from the softening moment resistance model is only few percents larger than the corresponding one resulting from the ideally plastic model in the example case.

Introduction

Cold-formed thin-walled structures and/or structural elements has been introduced in Romania on a larger scale mainly after 1990. Cold formed thin-walled members are widely used in any type of roofing systems (made of steel, wood or partially reinforced concrete structures). Using such structural elements for a roof structural rehabilitation is not a cheap solution, and a careful financial decision must be considered. Plastic dimensioning allows full exploitation of the potential of the cross-section. The paper deals with dimensioning models of gravity loaded cold-formed thin-walled multi-span purlins of ***C-***, ***Z-*** or ***Zeta*** cross-section with continuous full lateral restraint to one flange.

According to [3] ***(M, θ)***-graphs must be derived from tests. Rotation capacity, interaction of bending moment and shear force and softening behaviour – ***(M, θ)***-relation – are studied in the internal support test [3], in which a simply supported beam is loaded with a point-load ***F*** at the mid-span. The span ***s*** is ***0.4L***, and it represents the portion of purlin between the points of contraflexure on each side of the internal support, ***L*** is the actual span of the purlin. Finite element analysis can be used in the simulation of the internal support tests in order to study the effect of the shear force to the bending moment capacity of the purlin and on the softening behaviour of the purlin at the internal support.

Ultimate state is reached, when mechanism appears: purlin yields in the span in addition to the yielding at the support. The relation between the bending moment ***M*** and the plastic rotation ***θ*** can be supposed to be either a softening one or ideally plastic (Figure 1). The ideally plastic model can be used in hand calculations, but, for the softening model, a computer analysis is required. The softening ***(M, θ)***-

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relation yields safe ultimate load for the full scale test structure. The constant bending moment at the internal support ***M*** is obtained from the softening ***(M, θ)***-relation by using the estimated maximum rotation in practice (e.g. 2°-3°).

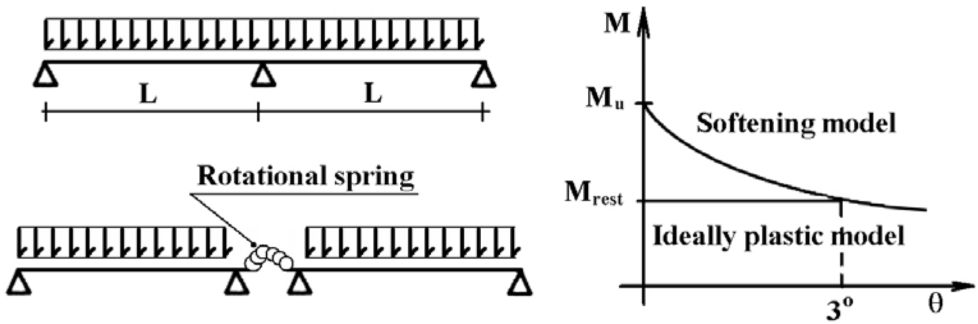


Fig. 1 – Example beam, softening moment resistance model and ideally plastic one

Example case

Analytical equations (1) and (2), which are derived for the case shown in Figure 1, are used in comparison of the plastic dimensioning models. Finite element method can be used in the computation of arbitrary multi-span continuous thin-walled beam with a softening moment resistance. A nonlinear rotational spring element can be used to define the ***(M, θ)***-relation of the purlin at the internal support. Suitable algorithms are presented e.g. in [5].

A continuous gravity loaded two-span purlin ***Z 250/2.0*** is used as an example. The span ***L*** is varied. The following properties were used: maximum bending moment in the span ***M =12,368 kNm***, effective second moment of area ***I=5,523,286 mm***^{4} and Young's modulus ***E=210,000 N/mm***^{2}.

Ultimate load $\mathbf{q}_{uip}$ can be evaluated using the ideally plastic model as follows:

$$q_{uip} = \frac{2}{L^2}\left(M_{rest} + 2M_{span} + 2\sqrt{M_{span}M_{rest} + M_{span}^2}\right) \tag{1}$$

where

M = constant bending moment at the internal support (***3,463 kNm***),

M = bending moment capacity in the span, and

L = span.

Corresponding rotation can be calculated from

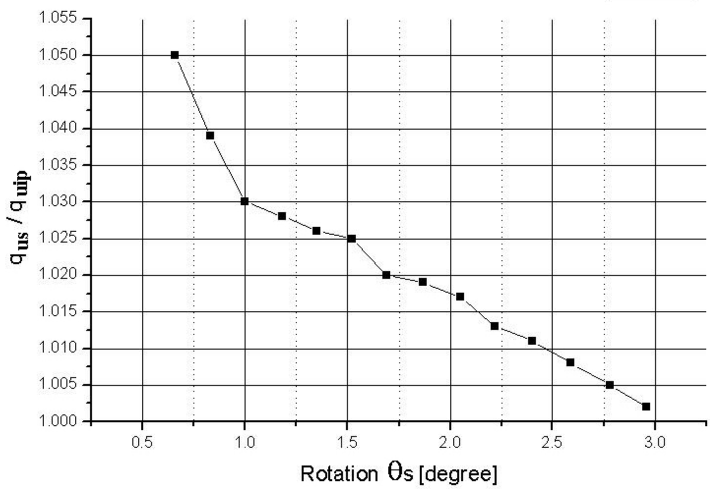
$$\theta_{ip} = \frac{L}{EI_{eff}}\left(\frac{q_{uip}L^2}{12} - \frac{2}{3}M_{rest}\right) \qquad (2)$$

where

E is the Young's modulus, and

I_{eff} is the effective second moment of area, respectively.

The softening moment resistance is taken into account by using ***(M, θ)***-relation instead of constant M_{rest} in equations (1) and (2) ***(M***M_u***=5,565 kNm)***. Ultimate load q_u by using the ideal plastic model is ***5.55 kN/m***, by using the softening model ***5.69 kN/m*** and the corresponding test gave ***7.90 kN/m***, when span ***L*** is ***4.5 m***. Ultimate load is labelled as q_{us} and the corresponding rotation as θ_s.



L [mm]	q_s [°]
2000	0.66
2500	0.83
3000	1.00
3500	1.18
4000	1.35
4500	1.52
5000	1.69
5500	1.87
6000	2.05
6500	2.22
7000	2.40
7500	2.59
8000	2.78
8500	2.96

Fig. 2 – (q_{us} /q_{uip}) -relation

Conclusions

The ideally plastic model overestimates plastic rotation and underestimates ultimate load in dimensioning compared with the ones yielding from the softening moment resistance model, but the difference in design is not significant.

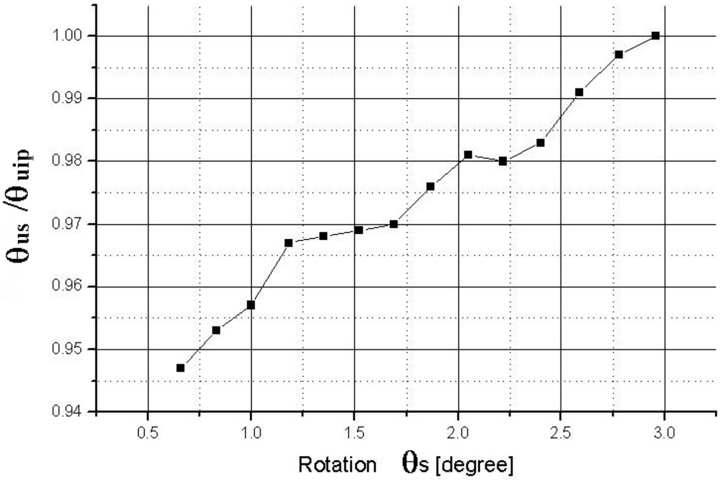


Fig. 3 – (θ_s / θ_{ip}, θ_s) -relation

L [mm]	q_s [°]
2000	0.66
2500	0.83
3000	1.00
3500	1.18
4000	1.35
4500	1.52
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SOFTWARE APPLICATIONS FOR THE ROAD ADMINISTRATION BASED ON TQM CONCEPT

Cristian DORNEANU[1]

Abstract

The works for the lines of communication are characterized by their linear specificity. In practice the length of this type of constructions is the element which raises problems as regards their administration and maintenance. A solution to these problems for administering a set of technical characteristics in very many sections, has been given by the utilization of these data bases.

Once the major changes of the society outlook, caused by the introduction of Total Quality Management concept, the necessity for participation of all factors implied in the conception, projection, realization, utilization and post utilization of a product of any kind is obvious.

Thus, in order to realize an administration and utilization of road constructions on the basis of TQM concept, first a proceeding is necessary so as to allow the access and understanding of technical data stored in data bases by all the implied factors.

Therefore, the realization of a software application is required so as to allow the virtualization and updating of data bases, on the basis of rights and permits given by their owner. Such application is described in this paper

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1. Introduction

As known, the electronic computer is involved in the constructions activity in three main trends:

- In the designing of strength structures, by using the finite element calculation
- In the elaboration of projects, by using the CAD software and texts editors
- In the financial administration and management of constructions, by using the data bases.

In every country it is an organization responsible for the national roads administrations, which have some technical-operational evidences rallied in different data bases. In Romania this organization is called National Administration of Roads, and the appropriate data bases collection form the so-called Central Bank of Road Technical Data. The function of this data bank is supposed to cover the following phases permanently:

- The collection of data: phase realized by means of specifically equipments and qualified staff in the activity of bridges and roads administration and maintenance
- Stocking of data: which in present is being made on Oracles data bases servers, that are maintained in front of all performance tests of the systems from this category
- Refunding/presentation of data: is realized by SQL inquiries, respectively the submission of data is made by a tabled text interface.

2. The choice of visualization method

Due to the fact that the tabled text interface does not fulfill the management in the decisional process, within Bridges and Roads Regional Division –Iaşi- the realization of a set of applications based on a Web interface has already started, by means of which the data contained in the data bases could be visualized.

Concerning the visualization part of data it has been made option for the realization of some JAVA 1/ applets which first of all has the advantage that they roll in a Internet browser (Internet Explorer or Netscape Navigator). The solution of browser has been chosen for two reasons:

- A “technical” reason. Due to the special scope Internet gained (especially the Web section) it can be observed the affinity of the main development software manufacturing societies (Microsoft, SUN, Borland, etc) for these technologies. That is why the Internet technologies has a very accelerating rate of development, compared to other technologies (which offer Stand – Alone applications) which do not benefit any longer of significant upgrades. In this regard it can mention the particular support which Java programming languages possess in the actual version of Oracle 9i server.

- A "technical" reason. Due to the Web development the browsers begin to become some applications more and more used and known by the most PC users, requiring thus a very slow learning cycle.

The choice for the submission of data by means of JAVA applets /1/ has been made taking into account the advantages offered by this solution:

- The utilization of JAVA technology offers a control of graphical interfaces at pixel level, which gives the possibility for some information to be presented visually as close as possible to the reality;

- Creation of some interfaces involving high mobility degree by means of a data base and easy to be handled by a road specialized staff, with medium training and even low in computer knowledge. Thus, the staff possessing road technical training has the possibility to detect the inappropriate data which may appear at a given moment in the data bank, due to the complexity of data bank structure and the big area on which the data are collected from.

- The easiness of JAVA language in transporting other Relations Data Base Management Systems and, also, the possibility for these to be used on Web. The application can be connected to two or even more data base systems, some summary changes being necessary during the application rolling, on the basis of the setting of the initial parameter.

- A final advantage, but not the least, is the free aspect of JAVA platform purchase and the license conditions, which offer the possibility for development and distribution of some application without any financial obligation.

4. The submission application

The modules of the application have been rallied and fitted in a unitary interface /2/, so as at the access of the launching folder the following interface is to appear:

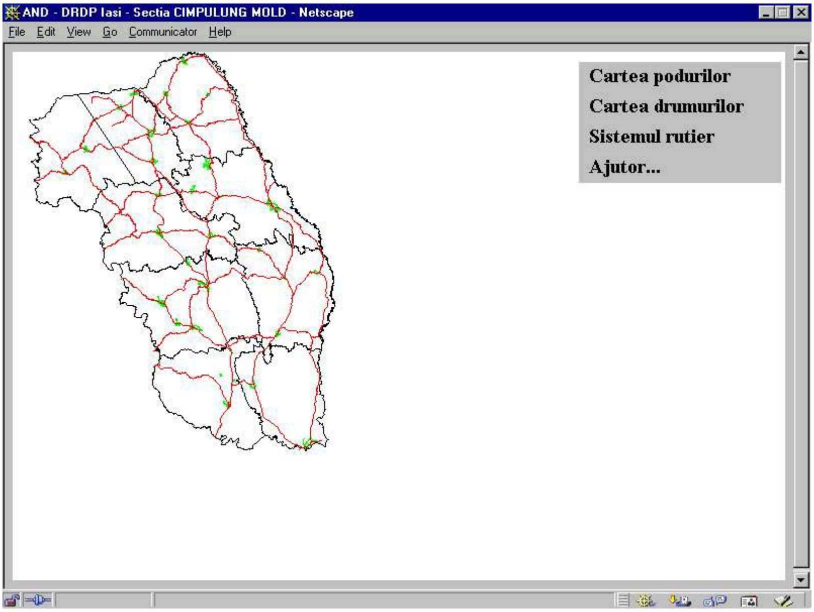

Fig. 1 – Main Menu

On the left side it can be observed a ladder drawing, where there have been figured certain elements, such as the county limits, national roads and cities.

On the left side it is simulated the function of a menu from which it can be selected the rolling of any application module:

1. The bridges book. It is an application strictly based on the accessing of html pages, which have been generated by using a servlet JAVA technology, implemented in a Tomcat server (which is also free). This module presents, for each bridge, certain data taken of the 7 tables from the data bank related to bridges, structured so as to offer a general image on the bridge. For each bridge, the relevant folder can be accessed from a road index. As a facility the printing is possible, for a bridge there are necessary over 3 A4-shaped pages.

Fig. 2 – Bridges book

2. Road book. This module is realized entirely in a Java applet, which presents the scaled road elements (width of carriageway, shoulders, ditches, parapets, markings, road signs, etc.) on a 150 meters length. There is the possibility to „parade” the road on the left-right side, by a 150 meters step, just pressing once by mouse the arrows buttons or to go directly to a certain kilometer while choosing the road and the kilometer from the ComboBox on the above right side:

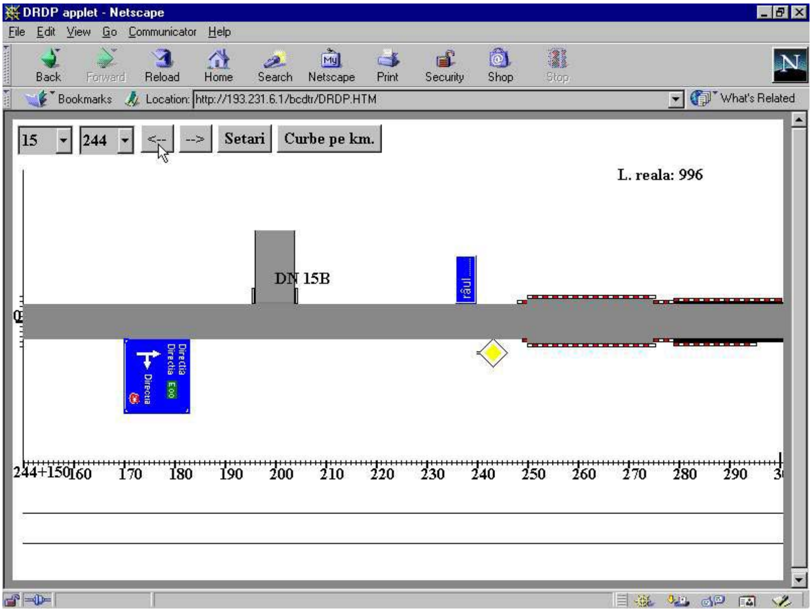

Fig 3 – Roads book

By a simple click mouse on different road elements there can be obtained information on those elements (road transversal section from meter to meter, junctions, culverts, bridges, etc.), everything is seen on the screen could be printed.

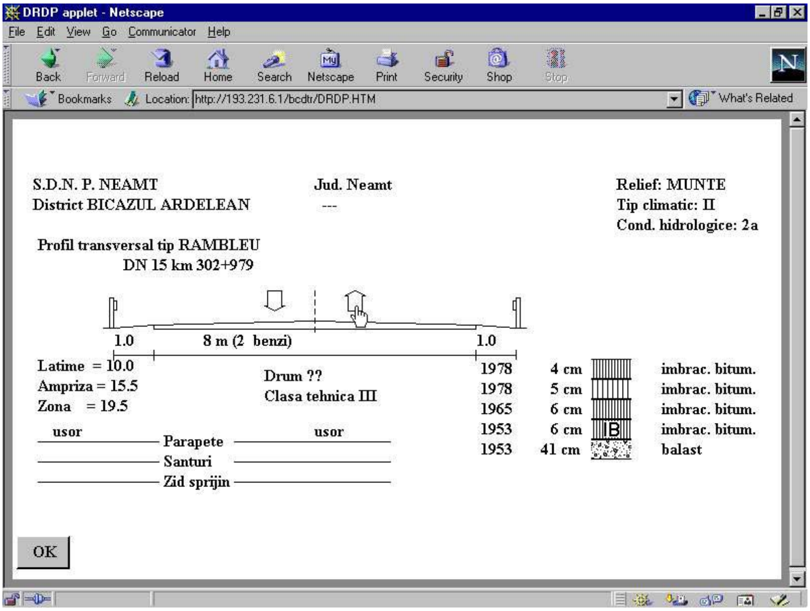

Fig. 4 - Roads book – Transversal section

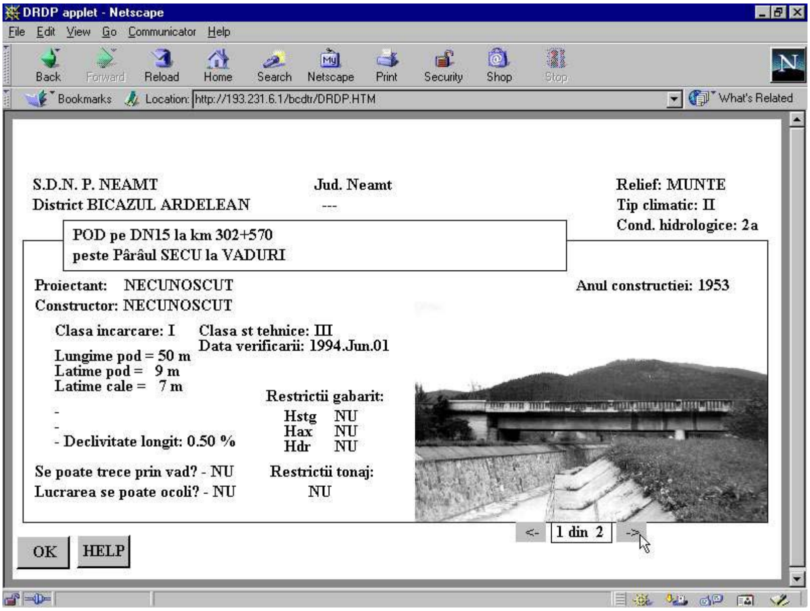

Fig. 5 - Roads book - Bridge

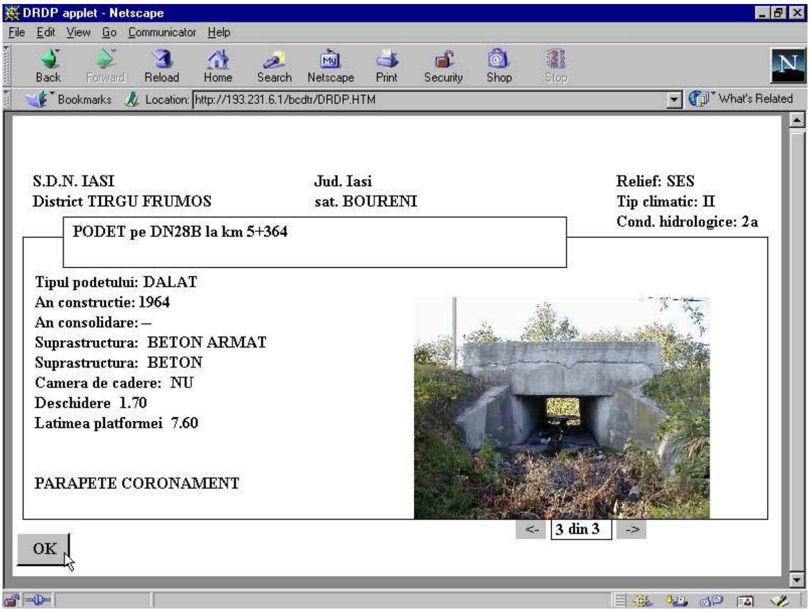

Fig. 6 - Roads book – Culvert

3. The road system. In the road book there can be obtained some punctual information (in a certain kilometer position) on the road system, but, the sectors involving homogenous road system within a road are initially impossible to be determined. For this reason it has been created the "road system" module where the homogenous sectors and their structure are easy to be detected.

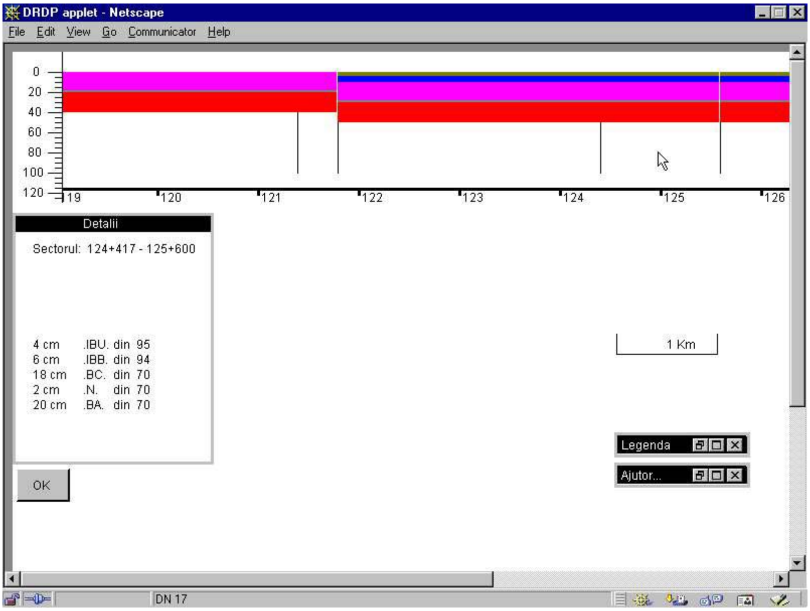

Fig. 7 – Road system

5. Conclusions

The advantages for the utilization of such programs consist of decreasing the decision time for the problems related to the road traffic by the fast access of requested information, the possibility for them to be located and the removal of wrong data included in the data bank, such as the easiness to update the evidences on written support.

By using the customer-client technology, the consultancy and update of data base can be made on network or Web, giving thus the possibility that the update could be made by the local employees and the consultancy from any location with Internet access.

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SOME CONSIDERATIONS ABOUT ACTIVE, PASSIVE, HYBRID AND SEMI-ACTIVE TECHNIQUES

Cristian PASTIA[1]

Abstract

Structural control has a long and successful history in civil engineering for mitigating dynamic hazards. The traditional approach to mitigate vibrations due to the earthquake and wind loads is to design structures with sufficient strength and deformation capacity in a ductile manner. This approach, based on the ensuring of strength-ductility combination, provides the strong wind or seismic action as ultimate load, accepting a certain number of structural or non-structural degradations. Usually, for a steel structure, the dissipation of the energy introduced in structure by dynamic action occurs only in the plastic hinges. For this reason, taking into account the way in which the load bearing structural elements of a steel system function together, a global plastic mechanism is generated.

New concepts of active control, designed in such a way that the control forces are supplied to the structure through the employment of the actuators, may exclude the inelastic deformations in the elements of the structural system. These systems require large external power sources that may reach several megawatts for large structure.

The promising alternative between the passive and active techniques has been developed recently in a form called semi-active technique. In another way said, semi-active control techniques make the rehabilitation of passive control systems, while they can occur similar performances of active control systems.

Hybrid control techniques blend passive and active control techniques. In the scheme of hybrid control the forces generated by the actuators are aimed to increase the efficiently of passive control devices.

Attention of this paper is focused on passive, active, hybrid and semi-active control systems. If passive control systems are used for enhancing the structural damping, stiffness or strength, the other control techniques employ controllable forces to add or dissipate energy in a structure, or both, due to the specific devices integrated with sensors, controllers and real-time processes to operate. This paper includes the advantages of these technologies in the context of dynamic hazard mitigation consisting of the following section: section 1 as an introduction; section 2 deals with passive control system; section 3 deals with active control techniques; section 4 deals hybrid control techniques; section 5 deals semi-active control techniques; and section 6 deals with general conclusions.

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1. Introduction

The modifications of the structural systems carried out in order to reduce vibrations have conducted towards the "Structural control" concept. The concept of structural control was the first time introduced by Yao (1972). This means that the structure is regarded as a dynamic system whose response variables (displacement, velocity and acceleration) are functions of time and in which some mechanical properties, typically the stiffness and the damping, may be adjusted to minimize the dynamic effects of load under an acceptable level. During the past decades many techniques have been proven to develop successful physical, analytical, numerical and experimental models in predicting the dynamic behaviour of the civil engineering systems that are subjected to excitations. According to the natural complex mode shapes of a structure and their corresponding damping values, we must find a good way to reduce the magnitudes of frequency response with respect to the excitation input. Frequency response analysis and transient problems became common design criteria for the community engineering. During the first half of this century the research directions have included the knowledge of a large number of diverse disciplines as follows: computer science, system theory, material science, sensing technology, stochastic processes, etc; some of which are not within the domain of traditional civil engineering (Spencer and Sain 1997). Therefore, in recent years, it has been paid a considerable attention to new concepts of structural control including a large variety of techniques that can be defined in four classes: passive, active, hybrid and semi-active. From historical point of view, passive control techniques such that base isolation and passive control devices are the first of them implemented. A lot of researches have studied structures equipped with these passive techniques and a lot of practical realisations have already implemented in many countries. These devices do not need an external power source and they are more economic and easy in applications.

Active, hybrid and semi-active control techniques have been studied extensively from a theoretical, numerical and experimental point of view for their performances (Karnopp 1974; Păuleț-Crăiniceanu 1999; Soong and Spencer 2002; Kobori 1999; Dyke and Spencer 1997; Preumont 1997; Magonette et al. 1999, 2001; Marazzi 2003; etc). Nowadays, civil structures considered structural systems that can comprise one or more control techniques. The four control techniques may be defined as follow:

2. Passive control

A passive control system consists of one or more devices, attached or embedded to a structure, designed to modify the stiffness or the structural damping in an appropriate manner without requiring an external power source to operate, developing the control forces opposite to the motion of the controlled structural system (Marazzi 2003).

Control forces are developed as a function of the structural response at the location of the devices (see Figure 1.1). Passive control may depend on initial design of the structure, on the frictional contact between elements of the structure or on the use of the contact dampers at the joints in structure (Budescu et al. 2001).

From the energetic point of view the passive control systems are divided into two classes:

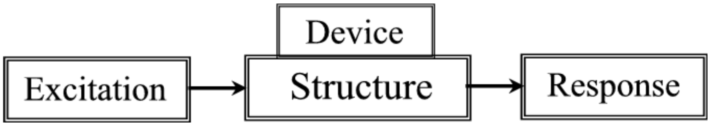

Figure 1.1: Components of an passive control system.

Base isolation. Isolation dampers such as elastomeric bearings or sliders (metal blocks), as well as isolation layers as fine sand or graphite material are introduced between the foundation and superstructure. Consequently, the *reducing of the input energy* of an earthquake in superstructure as well as the increasing of displacements across the isolation level are achieved due to the flexible decoupling between superstructure and foundation (Ciupala 1998). The most common adopted technique is the laminated rubber bearing with alternating layers of rubber and steel. The stiff steel plates provide lateral constrain of each rubber layer when the bearing is subjected to vertical load, but does not constrain the horizontal shearing deformation of the rubber layers. This produces a bearing that is very stiff in the vertical direction and very flexible in the horizontal direction.

Passive control devices. The control passive devices generally *dissipate energy inputted to a structure*. The motion of the structure is utilized to produce a relative motion within the passive control devices thereby the energy is dissipated. They may be also divided in two classes: energy dissipating devices, which are independent of the natural frequencies of a structure for their design, and tuned or resonant devices, which are dependent of the natural frequencies. An exception of frequency-independent devices are the viscoelastic dampers. Most of dissipating devices known as friction damper, hysteretic damper, viscoelastic damper or fluid viscous damper operate on principles such as frictional sliding, phase transformation in metals, deformation of viscoelastic solids or fluid orificing (Magonette et al. 1999). The plastic hinges are created in the structure when the elements of the structure are designed as energy dissipating devices.

The second class includes tuned mass damper (TMD), tuned liquid damper (TLD), tuned liquid columns damper (TLCD), suspended pendulum mass damper, mass pump, and so on. TMD and TLD systems have been extensively studied from point of theoretical, numerical and experimental view to control mostly wind input vibrations. Generally, inertial mass is attached near the top, through a spring and a viscos damping mechanism (e.g. fluid damper or viscoelastic damper).

The achievement of dissipative zones through structural, viscous and hysteretic, fluid damping must not affect the stability of the structure. Base isolation and passive control devices have been widely used in a number of civil structures in the Europe, US, Japan, Taiwan, New Zealand etc.

3. Active control

An active control system is defined as one in which a large external power source or many, from tens kilowatts to several megawatts, control actuators that apply forces to the structure in a prescribed manner (Marazzi 2003).

Such active control schemes are the active mass driver system (AMD), the active tendon system and the active bracing system (Kabori 1999). These forces can be used to both add and dissipate energy in the structure. The control forces within the framework of an active control system are generated by a wide variety of actuators that can act hydraulic, pneumatic, electromagnetic, piezoelectric or motor driven ball-screw actuation. The controller (e.g. a computer) is a device that receives signals from the response of the structure measured by physical sensors (within active control using feedback) and that on basis a pre-determined control algorithm compares the received signals with a desired response and uses the error to generate a proper control signal (De Silva 2000). The control signal is then sent to actuator. In feed-forward control, the disturbance (input signal), not the response (output signal), is measured and used to generate the control signals. Both feedback and feed-forward principles can be used together in the same active control system.

The cables are efficiently structural elements used in suspension bridges, cable-stayed bridges or other cable structures but they have the disadvantage of the great flexibility and low damping. The active tendon control systems based on damping techniques has been proposed to mitigate cables vibrations by many researchers in the recent time. Damping techniques consisting of a tendon actuator collocated with a force sensor were analysed and widely tested at ELSA (European Laboratory for Structural Assesment) on a large-scale cable-stayed mock-up (Magonette et al. 1999; Marazzi and Magonette 2001). The tested structure is a model of a cable-stayed bridge, equipped with two actuators on the two longest stay-cables. Due to the tendon actuator actively controlled the results show an important reduction in vertical displacement regarding the deck and a damping of whole structure increasing more then ten times when the bridge is subjected to an excitation. Consequently, the fatigue effects are mitigated. These technologies can be directly applicable to the real structure by scaling up the devices.

An active control system has the disadvantage of power failure during vibrations and great costs to implement such a technology. Such devices are independent of the natural frequencies of a structure for design.

4. Hybrid control

A hybrid control system is defined as one that implies the combined use of active and passive control system (Marazzi 2003).

A hybrid control system consists of employment of an active control device to improve and supplement the performance of passive control system. Alternatively, the passive devices embedded in a structure can decrease the amount of required energy if an active control system is installed in that structure. For example, a base isolation system can be improved using actuators that act to decrease the displacement of structure or a structure equipped with passive damping devices supplemented upon the its top with an active mass damper in order to enhance reduction efficiency of imputed vibrations.

Essential difference between an active and hybrid control system is the amount of external required energy to generate control.

A better control system using a less energy amount than active control system is hybrid mass damper (HMD) that combines a passive TMD and an actuator. Another difference is that a hybrid mass damper depends on the natural frequency of a structure whereas an active mass damper doesn’t depend on the natural frequency. These devices are similar to a tuned suspended mass damper or tuned mass damper with the exception that an actuator attached to the tuned mass can dynamically extend the amplitude of natural motion of the TMD. The designs of HMDs configuration include (Jerome 1998): multi-step pendulum HMDs, for example, one is installed in Landmark Tower in Yokohama, the tallest building in Japan; roller-pendulum HMDs, for example, arch-shaped HMD or V-shaped HMD are devices designed to behave like a mass pendulum fashion or passive TMD upon which sits a active mass driver (see Figure 1.2). The active mass driver at top of the tuned mass provides the necessary force to speed up the motion of the tuned mass at the start of the loading and provides a braking force at the end of the loading.

The energy and forces required to operate a typical HMD are far less than those associated with a fully active mass damper system of comparable performance, thus being more economical (Spencer and Sain 1997).

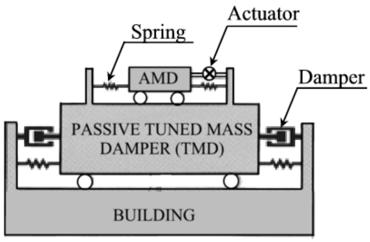

Figure1.2: Concept of HMD with a passive TMD upon which sits an active mass driver.

5. Semi-active control

A semi-active control system is defined as one that needs energy only to change the mechanical proprieties of the devices and to develop the control forces opposite to the motion of structure (Marazzi 2003).

Semi-active control systems are a class of active control systems for which the external energy requirements are smaller amounts than those of typical active control. A battery power, for instance, is sufficient to make them operative. Semi-active devices cannot add or remove energy to the structural system, but can control in real time parameters of the structure such as spring stiffness or the viscous damping coefficient. The stability is guaranteed, in the sense that no instability can occur, because semi-active devices use the motion of the structure to develop the control forces. A semi-active device will never destabilize a structural system whereas an active device may destabilize a structural system even though it has a low energy demand. These control devices are often viewed as controllable passive devices. Examples of such common semi-active devices can be categorized as following: semi-active hydraulic devices, variable stiffness devices, controllable friction dampers, controllable fluid dampers, semi-active tuned mass damper, semi-active tuned liquid damper and variable-orifice tuned column liquid damper. The variable orifice damper is the common device of the semi-active hydraulic devices. The device, described by Marazzi (2003), consists of a fluid viscous damper combined with a variable orifice on a by-pass pipe containing a valve in order to control the reaction force of the devices. The damping characteristics of a variable orifice can be controlled between two damping values (low damping when the valve is completely opened and high damping when the valve is completely closed) by varying the amount of flow passing through the by-pass pipe from one chamber of the piston in the other. In the intermediate positions of the valve opening process, the device produces a specific damping dissipation. The adjustment of the valve can be made usually electromechanically (e.g., servo valve or solenoid valve). The controllable fluid dampers are based on magnetorheological or electrorheological fluid that changes it's viscosity very quick in the presence of an adjustable magnetic or electric field (Dyke and Spencer 1997).

Such devices are independent of the natural frequencies of a structure, excepting the variable stiffness devices that can make an auxiliary stiffness element to be active so that structural system can be changed to realize non-resonant states to dynamic hazard mitigation.

6. Conclusion

The overall objective of this paper is to introduce the basic concepts of the passive, active, hybrid and semi-active systems. It's obviously that active, hybrid and semi-active control techniques consist of a number of important components as sensors, controllers, actuators and power generators that must be part integrated into the structural system. Due to the wide developments in the active control field the active control systems represent a future potential research and the practical application is one of big concern worldwide. These systems must be analysed regarding the stability, effectiveness, cost and required energy consumption.

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DESIGN CRITERIA AND ANALYTICAL RESULTS FOR YIELDING ENERGY DISSIPATORS

Septimiu LUCA[1]

Abstract

The introduction of energy dissipating devices as passive vibration controllers in structural systems under earthquake excitation is intended to reduce structural deformation and in certain cases to reduce the risk of damage to equipment attached to the structural system. Viscoelastic, frictional and elastoplastic elements are among the most commonly employed components of energy dissipating devices. All these devices provide the desired damping capability to the structural system and in certain cases increase the stiffness of the structural system.

In this paper the generic criteria and numerical simulation are presented for designing yielding devices, including a description of the engineering characteristics of the devices, and a description of important device-to-structure connectivities and flexibilities.

The engineering characteristics of yielding devices, i.e. the yield force, F_y, yield displacement, Δ_y, strain-hardening ration, SHR, initial stiffness, K_d, and ductility ratio, $u=\Delta_{max}/\Delta_y$, are defined in Section 2. Furthermore, there are described the various elements that are typically involved for supplemental damping application and the manner how they are related.

In Section 3 is presented a comparison of the models behavior with and without metallic yielding devices for 1 DOF and 3 DOF. Analyses were carried out using the single-story model (three-story model) and the commercially available computer program SAP-2000 (Computers and Structures Inc.2000). The Nllink element is used in SAP-2000 to model local structural non-linearities which only exhibit during nonlinear time-history analyses. The computer model is subjected to 12 seconds of ground motions (El Centro).

Many numerical simulations were performed for different values of the yielding force, F_y. In order to compare the results, the initial elastic stiffness of the device is described as a product of the elastic frame stiffness, SR*K_s, and the yield force of the device, F_y, is described as a fraction of the story-shear, FR*F. In order to compare single-story model and three-story model, the ratio of the interstory drifts of the unprotected model to the interstory drifts of protected model, FRD, and the ratio of the story shear of unprotected model to the story shear of protected model, FRF, are defined.

Finally, Section 4 presents the conclusions emerging from the cases studies taken up in this paper.

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1. Introduction

Extensive research and initial implementation have shown that passive energy dissipation devices can significantly improve the dynamic response of structures. These devices, which are also known as passive control devices, supplemental dampers, or passive dampers, can reduce structural responses due to wind, earthquake, and other dynamic loads. Such devices can absorb part of the energy induced in the structures, minimizing the energy dissipation demand on the primary structural members, and thus reducing the interstory drifts and minimizing non-structural damage. In addition, they can be designed to provide additional stiffness to the structure and to be easily replaced if they are damaged. However, seismic energy dissipation is a relatively new technology and there are many design-related issues that require additional research.

2. Engineering Characteristics of Yielding Devices and Device-Structure Connectivities

The principal parameters (Figure 2.1) that characterize the energy dissipation capacity of yielding device are the yield force, F_y, yield displacement, Δ_y, strain-hardening ration, SHR, initial stiffness, K_d, and ductility ratio, $u=\Delta_{max}/\Delta_y$.

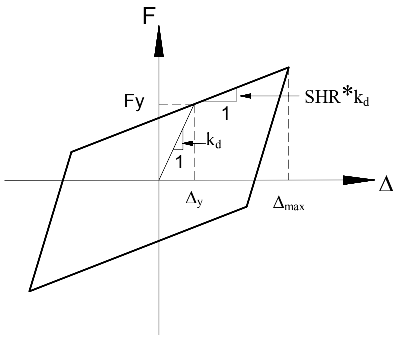

Figure 2.1 Idealized Device Hysteresis Loop

For most applications of supplemental damping devices, story drift deflections are quite small and deformations in the components that drive the devices, e.g. bracing, can be important. The most convenient and general way to account for these deformations is to explicitly consider all of the stiffness of the elements involved in the response of the structure. Figure 2.1 is a typical structure frame bay showing the various elements that are typically involved for supplemental damping application. The frame system could be just a single story frame as shown or it could represent a single story in a multi-story building, [1].

It is important to know that the damping device stiffness and the brace stiffness are series connectivity and the combined brace and damper stiffness is in parallel with the stiffness of frame [1].

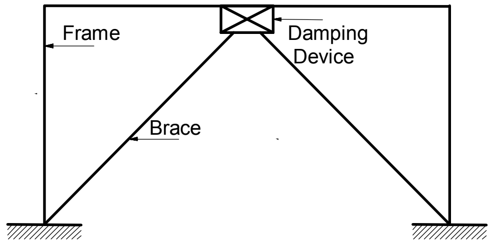

Figure 2.1 Typical Structure Frame with Damping Device and Bracing

Two important parameters used in design of yielding devices will be established: the combined device and brace stiffness, K_{bd}, and the total system stiffness, K_t.

$$K_{bd} = \frac{K_b K_d}{K_b + K_d} = \frac{K_d}{1 + \left[\frac{1}{K_b / K_d}\right]} \qquad (2.2)$$

$$K_t = K_s + K_{bd} \qquad (2.3)$$

Another parameter, SR, [1] used in the design of yielding device is the ratio of the horizontal combined brace and device stiffness (K_{bd}) to the frame structure stiffness (K_s). These stiffness and ratios have been introduced to describe only initial elastic values.

$$SR = K_{bd} / K_s \qquad (2.4)$$

The importance of the ratio of the brace stiffness to the device stiffness (K_b/K_d) varies significantly, depending on the specific supplemental damping device and the permissible magnitude of the interstory drift. The specific importance of the K_b/K_d ratio is revealed by considering extreme values for the brace stiffness (K_b). If K_b is very large, the device deformation is equal to the interstory drift deformation and the energy dissipation is maximized. If K_b is very small, the device is not deformed and there is no energy dissipation. A value of $K_b \approx 2K_d$ has been found to be practical for the design of yielding devices [1].

The strain hardening ratio, SHR, is typically small (from 0.01 to 0.1) and it’s effect on the response of structures with yielding device dampers is to reduce the lateral deformations and to increase the force in a structure.

3. Analytical Results

Analyses were carried out using the single-story model (three-story model) and the commercially available computer program SAP-2000. The computer model is subjected to 12 seconds of ground motions (El Centro) depicted in Figure 3.1.

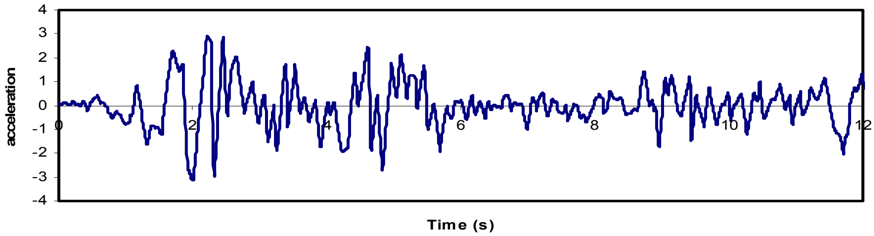

Figure 3.1 El Centro Ground Motions

The adequate parameters of analyses were chose according to eqs. 2.2, 2.3, 2.4 and many numerical simulations were performed for different values of the yielding force, F_y. In order to compare the results, the initial elastic stiffness of the device is described as a product of the elastic frame stiffness, $SR*K_s$, and the yield force of the device, F_y, is described as a fraction of the story-shear, FR*F.

A comparison of the model behavior without metallic yielding device and the model with yielding device is presented in Table 3.1 and Figure 3.2. In order to compare single-story model and three-story model, the ratio of the interstory drifts of the unprotected model to the interstory drifts of protected model, FRD, and the ratio of the story shear of unprotected model to the story shear of protected model, FRF, are defined.

Table 3.1 Engineering Characteristics of the Yielding Devices and the System Response

No.	**k_s (kN/m)**	**k_d (kN/m)**	**F_y (kN)**	**Relative Displacement (m)**		**Shear base (kN)**		**FR**	**FRF**		**FRD**	
				+	-	+	-		+	-	+	-
1	49091			0.008	-0.01	492	-388					
2	49091	140000	295	0.002	-0.0017	251	-272	0.599	0.5101	0.552	0.2	0.17
3	49091	140000	240	0.002	-0.0016	242	-263	0.487	0.491	0.534	0.2	0.16
4	49091	140000	200	0.00203	-0.0016	233	-254	0.406	0.4735	0.516	0.203	0.16
5	49091	140000	180	0.00207	-0.0016	228	-252	0.365	0.4634	0.512	0.207	0.16
6	49091	140000	140	0.0021	-0.0014	195	-237	0.284	0.3963	0.481	0.21	0.14
7	49091	140000	100	0.0026	-0.0017	180	-233	0.203	0.3658	0.473	0.26	0.17
8	49091	140000	80	0.0028	-0.0023	200	-222	0.162	0.4065	0.451	0.28	0.23
9	49091	140000	60	0.0032	-0.0035	239	-221	0.121	0.4857	0.449	0.32	0.35
10	49091	140000	50	0.0036	-0.0042	265	-233	0.101	0.5386	0.473	0.36	0.42
11	49091	140000	40	0.004	-0.005	293	-248	0.081	0.5955	0.504	0.4	0.5
12	49091	140000	30	0.0048	-0.0059	329	-272	0.060	0.6686	0.552	0.48	0.59

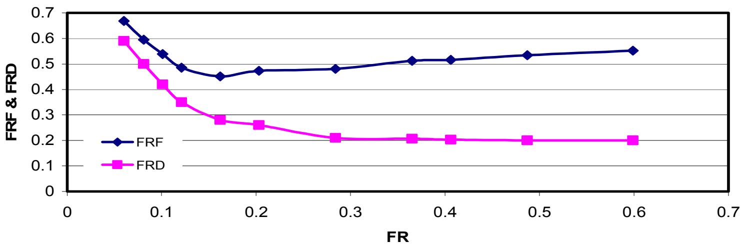

Figure 3.2 Maximum Response of Single Story Model with and Without Metallic Yielding Devices

Figure 3.3 shows the force-displacement relationship in the metallic yielding device for cases no. 6, 7, 8 from Table 3.1, also the interstory drift time histories for case no. 8 and case no. 1(unprotected single story model).

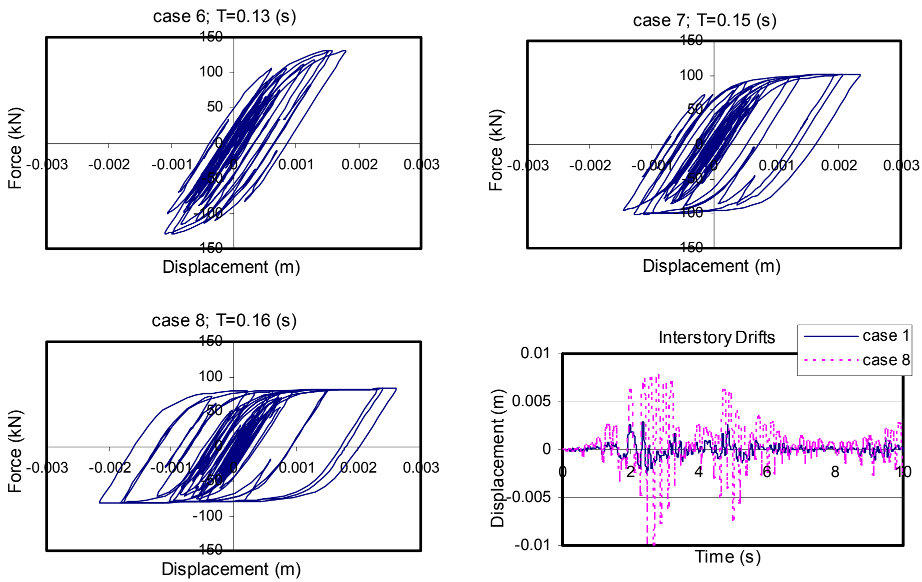

Figure 3.3 Loops of Force vs. Displacement of Yielding Device

The presence of some energy dissipation in conventional buildings has been recognized from long time ago and accepted in engineering applications. Although the nature of the energy dissipation inherent in buildings has not been explicitly elucidated, inherent equivalent viscous damping in the range from 2% up to 5% of critical has become accepted in practice for linear response analysis of typical buildings.

An equivalent viscous-damping factor ξ_h can be defined to take into account the hysteretic damping of the supplemental device. Any inherent viscous damping factor ξ_i of the model must be added to ξ_h to obtain the effective viscous damping factor ξ for the system. Thus,

$$\xi = \xi_i + \xi_h = \xi_i + \frac{2 \cdot A_h}{\pi \cdot A_r} \qquad (3.1)$$

where A_h is area of the hysteresis loop and A_r is area of the minimum rectangle which contain the hysteresis loop, [2].

It can be noticed from Table 3.2 the value of effective viscous damping factor, ξ, calculated with above relation for cases no. 6, 7, 8. The relationship of relative displacement vs. shear base and maximum hysteresis loops that were used to compute the equivalent viscous damping factor, ξ_h, are shown in Figure 3.4

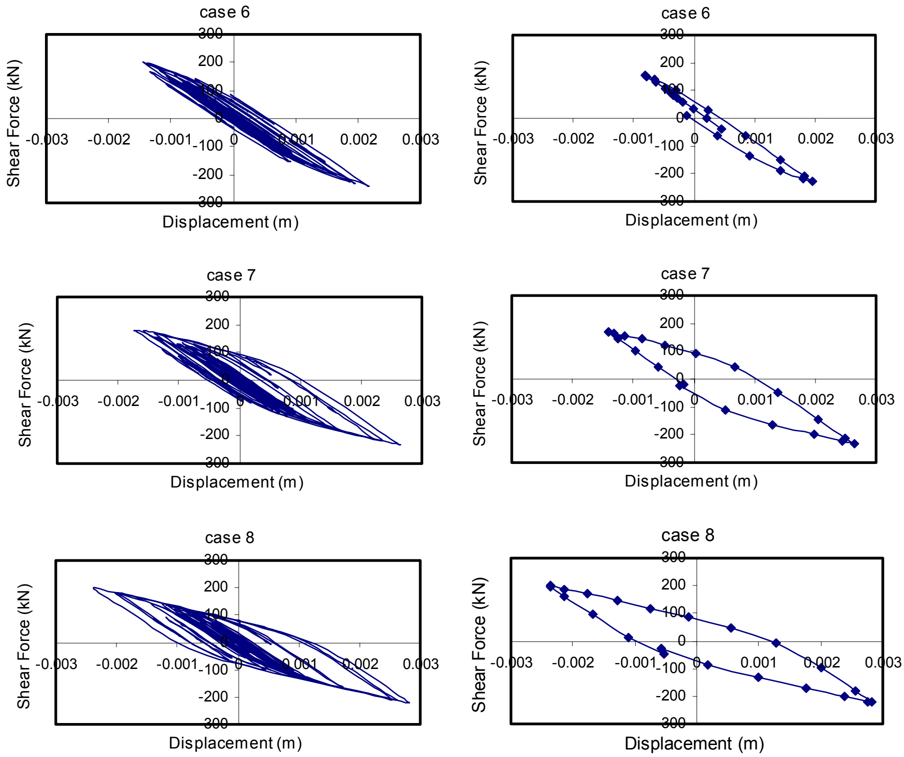

Figure 3.4 Shear Force vs. Displacement Loops of the Single Story Model

Table 3.2 Effective Viscous Damping Factor, ξ

No.	k_d (kN/m)	F_y	K_{eff} (kN/m)	k_s (kN/m)	FR	SR	T (s)	Relative Displacement	Shear Base	ξ
6	140000	140	82987	49091	0.284	2	0.13	0.0021	237	0.13
7	140000	100	43506		0.203	2	0.15	0.0026	233	0.20
8	140000	80	31250		0.162	2	0.16	0.0028	222	0.21

It should be noted that the relationship expressed in equations 2.2, 2.3, 2.4, are also valid for multi story buildings. The frame system from Figure 2.1 could be just a single story frame as shown or it could represent a single story in a multi story building

Figure 3.5 shows the effect of the FR parameter variation on the system response FRF and FRD, namely. It also could be observed a comparison with the single story system above analyzed, and with another single story model which has the same period.

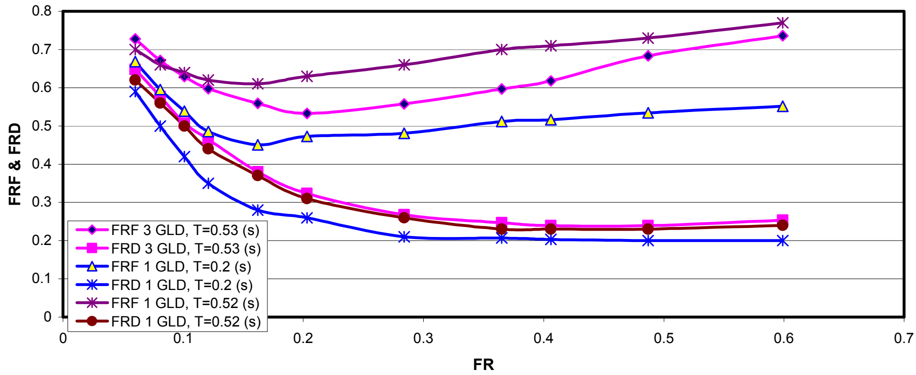

Figure 3.5 Maximum Responses of 3-Story Model and Single Story Model with and Without Metallic Yielding Devices

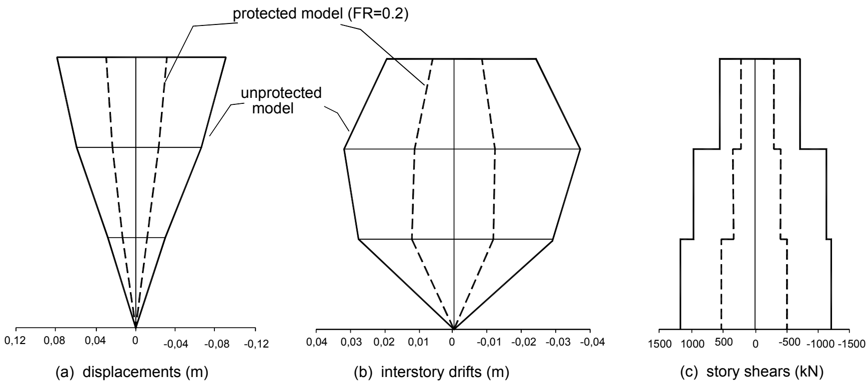

Figure 3.6 Response Envelope for 3-Story Model (FR=0.2)

The Figure 3.6 presents the response envelope in terms of displacements, interstory drifts, and story shears for three story model with and without yielding device. The time history variation of the relative top displacements for both protected and unprotected, 3DOF are shown in Figure 3.7.

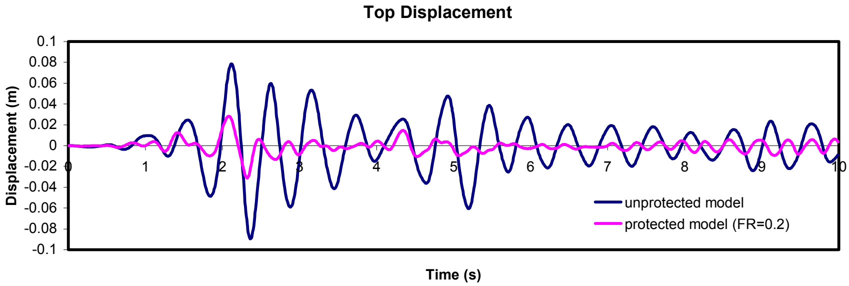

Figure 3.7 Relative Top Displacements of 3-Story Model

4. Conclusions

Increasing the damping of a building significantly reduces the acceleration and displacement response to earthquake ground motions and therefore, reduces the damage level and the potential of building collapse.

In this paper a parametric study was carried out to determine how the energy dissipation devices modify the structural response. The use of energy dissipation devices coupled with steel braces allowed to stiffen the building, i.e. reducing the natural period, and to dissipate energy when the devices start yielding. In some cases, the reduction of the natural period of the protected systems alone could improve its seismic behavior by moving the structure away from the critical period zone of the response spectra.

The passive energy dissipation device is seen to be quite effective in optimizing the dynamic performance of the investigated models. The enhanced energy dissipation capacity attributed to the hysteretic behavior of the yielding device is capable of suppressing the story drift to within the allowable limits. Working in tandem with the support brace system, the device can provide adequate lateral stiffness to resist wind load and moderate earthquakes. For each case analyzed the response of the protected systems has been reduced significantly when compared to the unprotected systems.

However, seismic energy dissipation is a relatively new technology and there are many design-related issues that require additional research. As the experience with energy dissipation systems increase, the results of the related research will be available and the design requirements will be refined, as a consequence.

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ENCALC – A SOFTWARE FOR THE EVALUATION OF THERMAL ENERGY CONSUMPTION IN BUILDINGS

Adrian DOLOCA[1]

Abstract

The paper presents a new software tool designed for the evaluation of the thermal energy consumption in buildings. The program enables a designer to input building elements with their characteristics (dimensions, structure, orientation), as well as climatic conditions and to calculate the necessary thermal energy for heating and warm water preparation. Results are shown in tables and charts and reports can be produced. The software is for architects and civil engineers in various situations like: thermal rehabilitation of the old residential buildings, schools, hospitals, and new building design. It is a user-friendly computer program, with a Romanian user-interface, compatible with Windows operating systems, easy to install and operate. The first chapter, is an introduction to the application areas of the software and the category of specialists that could benefit. The second chapter (*ENCALC Profile*) presents in more detail the characteristics of the program and the methods used in computation. The functions of the program are: testing the existing buildings if they comply with the current energy consumption standards; optimization of a building energy performance, in the design phase, by analysing various solutions; evaluation of energy conservation measures applied on existing buildings; evaluation of the necessary energy resources at the national level by calculating energy consumption for representative buildings. Also the simplifying assumptions on which the calculation is based are showed. The third chapter (*Working with ENCALC*) discusses the working session with ENCALC. The main window, types of projects, input of the characteristics of building elements, climate conditions, structure definition, etc. are presented. Next, the calculation and the output of the results (chapter 4) is considered. Suggestive screenshots are included to demonstrate the ease of use and the ability of the program to process a wide range of situations. Some conclusions are drawn in chapter 5 showing that ENCALC can assist the architects and building engineers in evaluating the thermal energy consumption in existing buildings and in designing new ones. This is required by a series of European as well as Romanian standards.

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1. Introduction

Globally, the building sector accounts for approx. 40% of the primary energy use in Europe, including Romania. As a consequence, it has an important contribution to the CO_2 discharges and to the economic balance at the country level but also at each building owner. Now, a series of laws and technical codes stipulate the necessity to establish the specific final energy consumption and to cut down this indicator from 200-250 Kwh/m^2·a to about 75-120 Kwh/m^2·a, if the energy is obtained from fossil fuels [1,2,3].

The building sector therefore stands in great need of practical instruments for energy consumption analysis to be used by the architects and civil engineers working for the:
- thermal rehabilitation of the old residential buildings, schools, hospitals, etc.;
- new building design.

The aim of our action, together with CalCon (an establishment of the Institute for Building Physics in Holzkirchen, Germany) was to develop a user friendly software adapted to the last requirements of technical codes in Germany and Romania.

2. ENCALC Profile

ENCALC computes the energy necessary for heating and warm water preparation. The calculation methods are those specified by the European Norm EN382 included also in the German Norm DIN EN ISO 13789 [4] which will be also implemented in Romania. They are based on the energy balance in stationary conditions but take into account the external temperature variations, the internal heat input and the heat coming from the sun radiation (fig. 1).

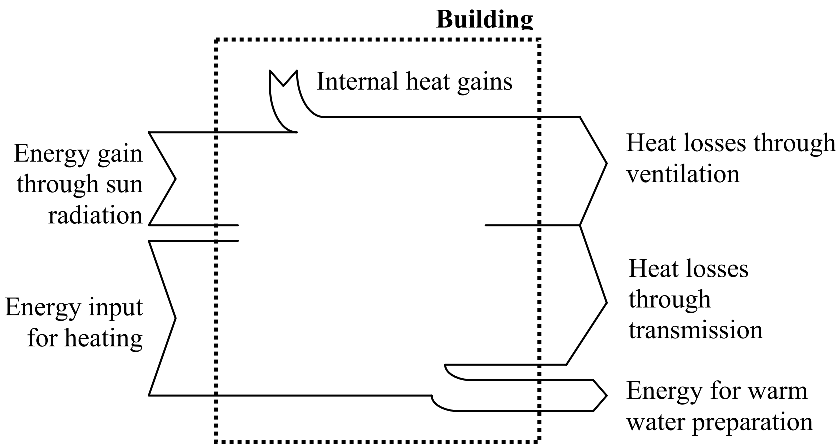

Fig. 1. Energy balance in a building

The software has the following applications:
a) testing the existing buildings if they comply with the current energy consumption standards;
b) optimization of a building energy performance, in the design phase, by analysing various solutions;
c) evaluation of energy conservation measures applied on existing buildings;
d) evaluation of the necessary energy resources at the national level by calculating energy consumption for representative buildings.

ENCALC uses a database which stores the characteristics of the building materials and structures in accordance with the EU and our country. It also contains climatic data for Romania.

The calculation method on which the software is based, uses a series of simplifications:
1) the temperature of the heated spaces inside the building is considered constant;
2) the thermal external bridges are not computed independently. Instead, their effect is taken into account by using the exterior dimensions of the building;
3) the influence of the mechanical ventilation systems is not taken into account;
4) a steady heating of the building is considered.

These hypotheses are acceptable for most cases and do not affect significantly the precision of the final results.

3. Working with ENCALC

ENCALC is a MS-Windows compatible program which is project oriented. The steps for creating the project and doing the computation are as follows.
- define the external geometry by sketching and inputting dimensions;
- define parameters of the building and the climatic conditions;
- input the building elements (type, orientation, dimensions, structure). Elements through which heat is transmitted must be input;
- start the calculations of the energy balance (fig. 1). Monthly values are computed first and then the annual values;
- display results in form of tables and bar charts;
- generate report in HTML format;
- test effects of thermal insulation layer thickness on final results;

In the next images, the ENCALC graphical user interface is presented.

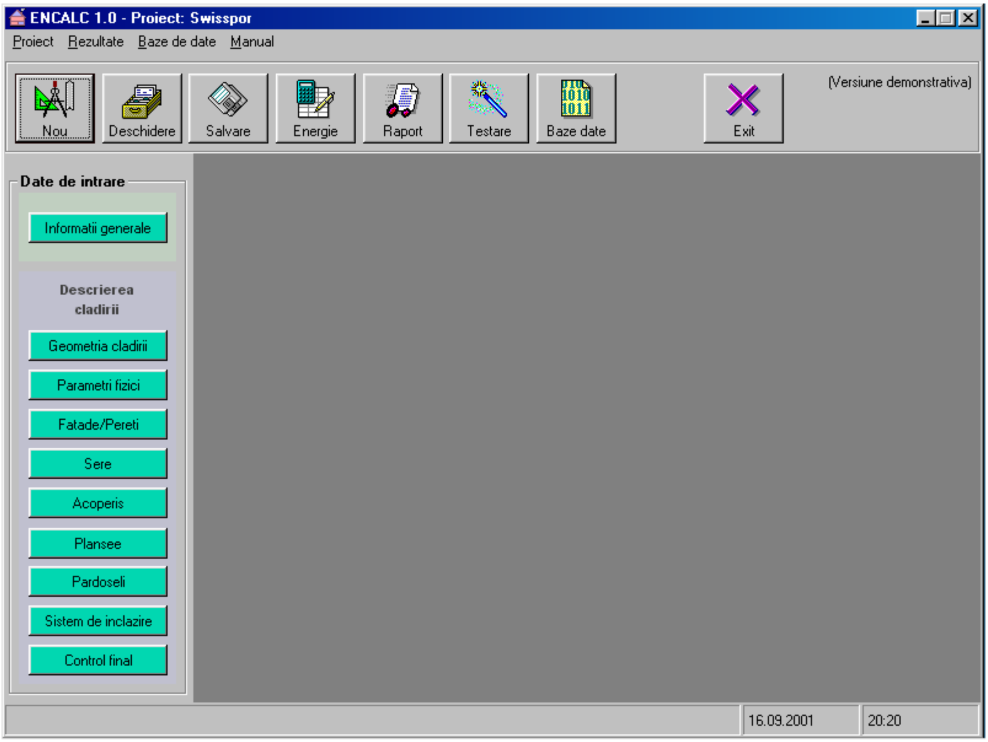

Fig. 2. ENCALC main window

The commands are grouped in a menu at the top side of the main window (fig. 2). Frequently used functions (opening a project, saving, energy calculation, reports, etc.) are available through the toolbar under the menu.

The left toolbar appears when a project is created and defines input data categories:

1) General information;
2) Building geometry;
3) Physical parameters;
4) Facades and walls;
5) Greenhouses;
6) Roof;
7) Decks;
8) Floors;
9) Heating system;
10) Final check.

When creating a new project, two options are available (fig. 3). The data concerning the building elements will be input by hand (option A) or a sketch will be drawn from which the dimensions will be automatically computed (option B).

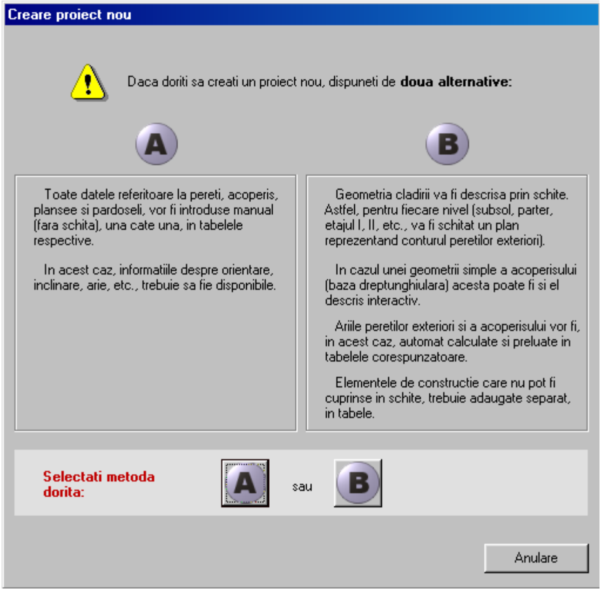

Fig. 3. Options for creating a new project

When using the second option, a sketch of each level must be drawn (fig. 4).

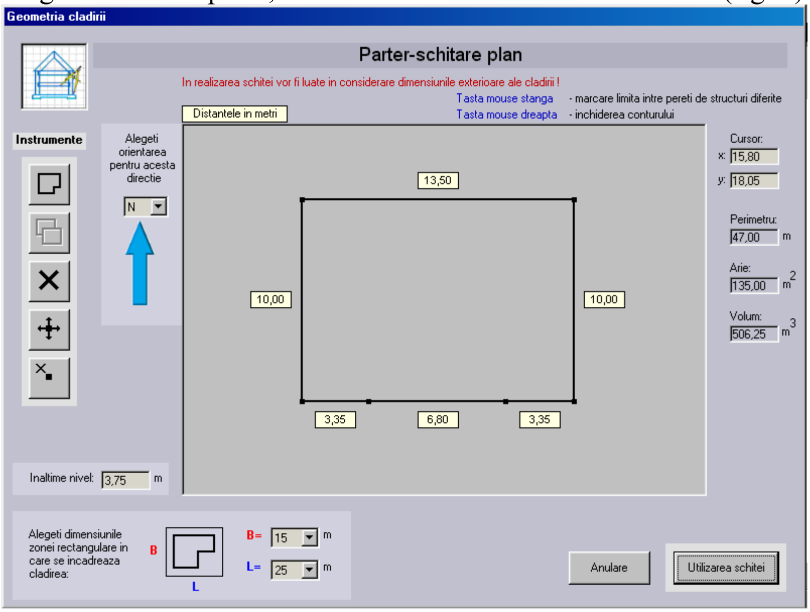

Fig. 4. Sketch of the ground level

Simple roofs can also be sketched (fig. 5).

Choosing a building element is very simple. Just click on the image or select it from the table below (fig. 6).

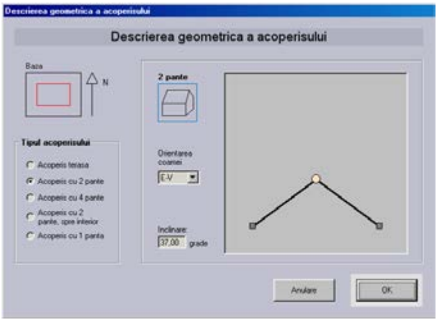

Fig. 5. Simple roof types

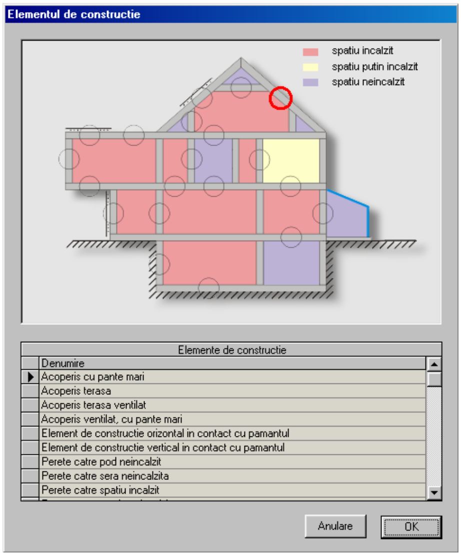

Fig. 6. Selecting building elements

After the building elements have been input they appear in a table like the one in fig. 7.

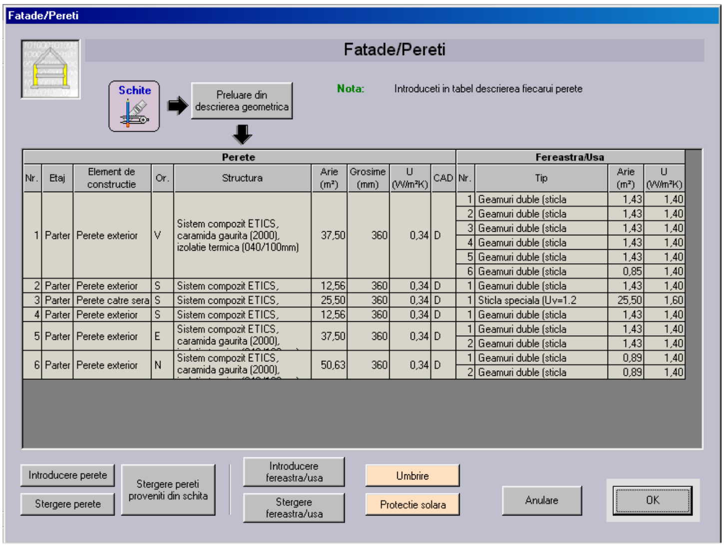

Fig. 7. Detailed definitions of building elements (walls in this case)

Selecting the structure of each element can be also easily done from the existing database (fig. 8).

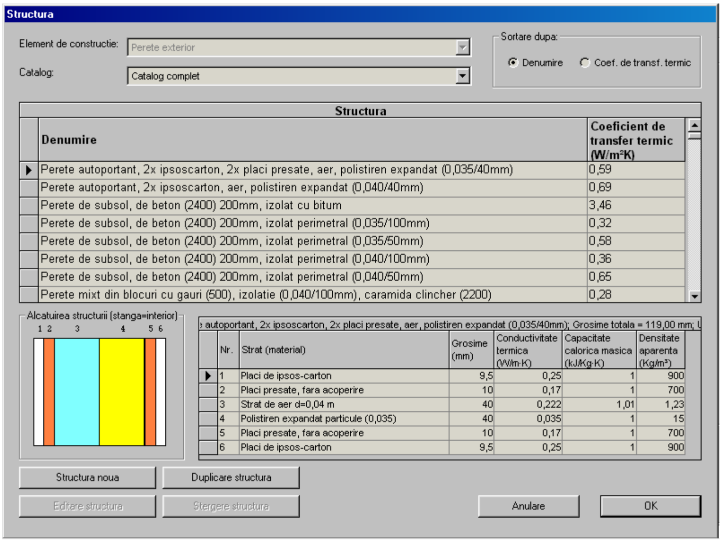

Fig. 8. Choosing a structure from the database

4. Presenting the results

Results of energy computation are presented as tables and bar charts simultaneously (fig. 9). These results are:

- energy consumption for heating without the warm water preparation;
- total energy consumption for heating and warm water preparation taking into account the efficiency of the heating system;
- heat gains and losses for each month and for building elements;
- values divided by the area and the volume of the inside space.

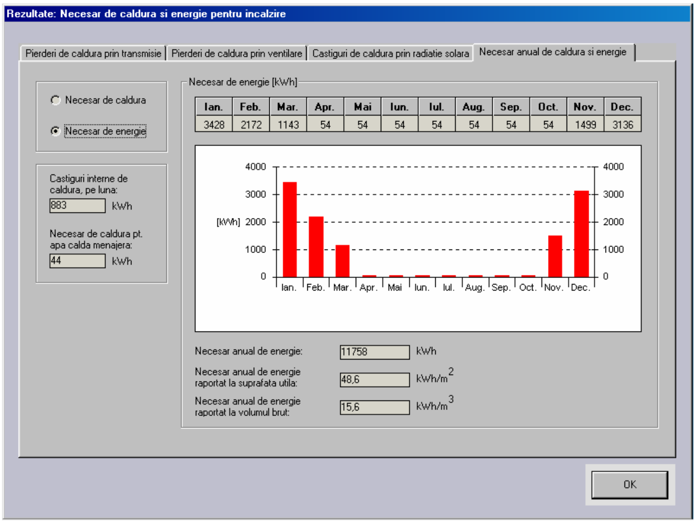

Fig. 9. Results

5. Conclusions

The ENCALC software which is available in Romanian, is a very useful tool for the design of the new buildings as well as for the modernisation of existing buildings which are required to meet the present energy conservation standards. It facilitates the work at the three sections of the thermal audit according the N-47 norm (the thermal expertise), N-48 (diagnosis and means of reducing the heat consumption), N-49 (the energy certificate).

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A CRITICISTIC POINT OF VIEW OF THE ASEISMIC DESIGN CONCEPT

Mihai BUDESCU[1], Ioan CIONGRADI[2], Octavian ROŞCA[3]

Abstract

In certain situations, some design codes provide increases of the computed seismic forces, in order to strengthen the serviceability safety under seismic loading, without any scientific reason.

There cannot be neglected the restrictions stated in the structural design codes that increase the structural efforts, limit the reinforcement ratios as a consequence of the ductility restrictions, limit the displacements, introduce a large amount of structural provisions in one way and reserves covering the designer's "conception" on the other way.

It becomes more intuitive to perform the seismic computation in the elastic domain, in such way the obtained efforts and displacements become a reference limit, easily interpreted by the structural engineer. In this way one may use differentiated coefficients of reduction for each effort type, element, member end, joint, anchorage, assembly, etc. This reduction can be carried out by multiplication by a subunit coefficient (of ψ nature), or by division by a coefficient greater than one (of ductility coefficient nature). Such a computation will underline the real system behavior and pays attention to the designer about the vulnerable areas. We appreciate this proposal to be necessary when the new aseismic design codes will be written.

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1. Overview

Almost all the aseismic design codes are based on a specific reduction of the seismic shear level, that has various reasons, yet difficult to be quantized. Obviously, this approach is based on practice, experimental tests and most of all, on criteria correlated with costs.

In certain situations, some design codes provide increases of the computed seismic forces, in order to strengthen the serviceability safety under seismic loading, without any scientific reason [1]. It is difficult to agree that a 20% or 40% increase leads to a similar increase of the structural safety index or to the desired purpose.

There are a lot of influencing factors in the process, starting with the evaluation of the seismic action up to the design stage of a structure. In this way, there cannot be neglected the restrictions stated in the structural design codes that increase the structural efforts, limit the reinforcement ratios as a consequence of the ductility restrictions, limit the displacements, introduce a large amount of structural provisions in one way and reserves covering the designer's "conception" on the other way.

Taking into account these interventions, it is difficult to state the final result i.e. the serviceability safety and structural reliability.

In this "network" of correlations it occurs the dangerous situation for the designer to misinterpret the results; therefore the designer finds reasons to grant or retain certain amounts of structural reserves. Such as approach is completely out of scientific basis, depending on the designer's experience.

Even though the design activity is submitted to a strong control, this activity provides only the check of the legal frame of the proposed goals without any intervention on the conception process of the project, i.e. the re-design.

Why do we bring into attention these facts, which may appear not important considering the circumstances of the precisely stated provisions? The response is found in the analysis of a large number of structural projects and discussions with structural engineers.

In these circumstances, we believe that synthesizing some observations, without pretending to cover the entire picture, may lead to a more attentive approach of the design process and especially to a better understanding of the aseismic design concept.

2. The model creation

Many things are written about the modeling process, but few people are taking into account the minimum amount of observations required to create an accurate structural model. This is due to the lack of knowledge of the accumulated experience in this field in one way, and of the ad-litteram code interpretation on the other way.

The theoretical model of a structure is nothing else but the representation of elements with the assemblage system, stiffness, inertia and eventually, the damping. The representation must also contain the support connections or connections with elements that might inter react when the structure is submitted to external loads.

Thus, in order to establish an appropriate computational model, one must carry out a system of operations, following a linear or cyclical path, depending on the way of model representation (all at once or progressively), the way of use and precision. These operations are:

- building of the computational model (structural sketch);
- attachment of a computational theory (idealization of the computational sketch);
- adoption of a computational method;
- choice of the appropriate numerical solver;
- solution;
- interpretation of the results.

The replacement of the real system by the computational model is based on the structural discretization (division, splitting, quantizing) in an amount of representative individual elements. In this process the attention is focused to structural discontinuities (geometric, inertial, material variations) support conditions, releases, load cases and load combinations, inter reactions, eccentricities, areas with important stresses, contact/ impact areas, etc.

There are no algorithms, neither general approaches to perform the transfer from the real structure to the structural model that will assure an unique model, that approximates with a certain given degree the real structure. That’s why it is possible for a given structure to obtain several models, all correct, but providing different performances, even just for a specific load case. The structural model is based on intuition, imagination and the previous experience of the engineer.

The model elements are structural (material points, rigid bodies, deformable bodies – bars, 2D and 3D elements) and connectors (non deformable – rigid links – pendulum and deformable – springs).

In case of static analysis the structure is decomposed in a finite number of constructive deformable elements; in case of dynamic analysis these elements must have the fundamental frequency large enough (greater than 30… 33 Hz) in order to be considered stiff and more than that, the mass positions (material points and/or rigid bodies) and energy dissipators (dampers, dash-pots) must be specified.

For the selection and statement of the seismic computational model it is necessary to follow some specific aspects, such as:

- the number and position of lumped masses shall respect the repartition of the real masses, as a measure of the inertia; in most cases the sum of the model masses must be equal to the sum of the structural masses; the positions of centroids must coincide;
- the use of 2D / 3D models as a consequence of the symmetry axes and the value of the torsion stiffness (natural mode uncoupling). The direction of the earthquake motion also implies the model selection in such a way that the simplifications must not lead to the lack of significant modes of vibration;
- the appropriate definition of the physical characteristics of the materials, especially the Young and shear modulus, the deformation modulus of the foundation ground, by taking into account the time behavior or strong motion effects, the structural element stiffness and the ground stiffness. Thus it is important to select a range of values for certain physical properties in the view of parametric analysis;
- The model must contain a sufficient number of DDOFs to represent correctly the different structural deflection shapes; usually one must compute all the eigenvectors for which the natural frequencies are lower than the rigid body frequency or those for which there is no significant dynamic amplification. For the usual multi storey buildings the number of necessary eigenmodes is reduced. Thus, for a regular structure that is computed by the means of a 2D model along each horizontal direction of motion only three modes of vibration are enough, the first – fundamental one – is far away the most important. If the same structure is computed by the means of a 3D model, nine modes of vibration will have to be taken into account (a group of three for each of the two horizontal directions perpendicular in plane and three modes for torsion about the vertical axis). One may notice that is necessary to apply some criteria for the selection of the significant modes, for instance the modal mass method (usually the sum of modal masses must be grater than 90% of the total structure mass).
- There are situations when the model experiences local modes of vibration, corresponding to the oscillations of a substructure, i.e. cantilevers, attic elements or flexible elements sustained by rigid elements. The selection of the eigenmodes by analytical methods (i.e. modal masses) is no more efficient in these cases, so the plot of modal shapes remains the appropriate way to find out the local modes. The occurrence of these modes may demonstrate the modeling errors or a weak conformation of the structure/part of structure that provides

local modes. In these situations it is worthwhile to analyze the possibility to eliminate the local modes by the means of the computational model, providing supplementary connectors and/or connecting elements, bracings, walls, shear walls etc.

- The representation of the non structural elements, that do not participate to the structural strength, such as closings, liquids or other products stored in certain types of containers (silos, bunkers, water towers, reservoirs etc.). Usually these elements are divided in two classes: those that “follow the motion” and might be represented by the inertial features only and those that are compulsory to be introduced with the inertial and stiffness characteristics. It is not always possible to decide just from the start if the structural retrofitting is benefit to the building safety, thus one may use many models in which the coupling effect varies inside some realistic limits. More than that, the stated coupling effect must be applied in practice.
- The model refining must be carried out according to the desired goal:

(i) a detailed knowledge of stresses/efforts in several elements or points of the model; in this situation the precision of the results is checked by comparison of two subsequent models with different partitions;

(ii) if a global response is needed, for instance in case of the “stick” model with equivalent stiffness that provides future information for detailed local analysis. Today equivalent models are used more and more for complex structures, or in case of a large number of DDOFs.

3. The design steps

The statement of the design steps starting from the “initial product of the architect” up to the design project needs series of corrections to meet the agreements between structural and architectural requirements. These elements are usually met in the design practice, but hide a tremendous process (see Fig. No.1).

Thus, the engineer creates the model by taking into account the above mentioned aspects plus the conditions imposed by the building place. Paying respect to codes, the designer will generate the loads and obtains the strains and stresses. This current practice may cause series of mistakes without any correct interpretation of the load-structure relationship, that in case of an inexperienced engineer leads to a wrong design.

It is worthwhile to mention the codes related to the cross-section design and assembly for several kinds of structures – r.c. frames, shear walls, steel structures etc. – based on the computational results. All these provisions impose arbitrary increases of efforts, that may cause serious distortions for the structural response under initial actions. A set of specific constructive measures will be finally applied, based on the designer's experience, thus resulting the design project.

This entire difficult process carries on the designer fingerprint, i.e. the reflections of his experience and intuition.

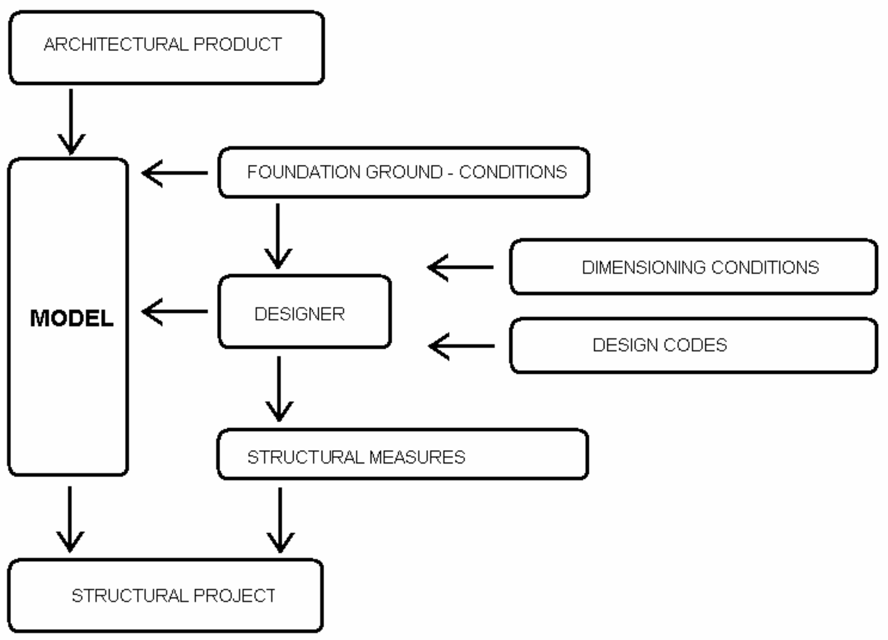

Figure No. 1

Not seldom, the difficulty of judging the codes and the mismatch between some dimensioning conditions and certain structural types may lead to wrong systems.

If we tackle this subject only in the view of the computation of the seismic response, we notice the reduction of the computed seismic forces according to the code [1], taking into account the bearing capacity and ductility reserves (the ψ coefficient); these reserves are really difficult to be quantized in practice. Afterwards, increases of the efforts when dimensioning the elements are imposed, based on the provisions of the codes [1, 2], thus leading to an abstract approach of the design process, hiding the real system behavior to the designer.

The most trivial example is the appearance of the tensile stresses in columns and beams (see Fig. No. 2), a consequence of the real seismic action without performing any reduction due to the ductility and other structural reserves. This phenomenon may often occur in case of tall buildings and when ignored by current design is leading to vulnerable areas. A stress such as tension is not susceptible to be redistributed to adjacent elements and carried on by the ductility reserves in extreme situations. In this situation the repercussion is: the designer doesn't know what happens inside the structure, therefore he doesn't take the necessary countermeasures and the product is conceptually wrong. A RC element subjected to tension may provide severe future repercussions after the concrete degradation.

This fact is more dangerous when the gravitational loads are over evaluated "consciously" due to the technological reasons, situations often present in case of industrial buildings.

If the designer has the real control over the structure, he would be able to delimitate himself the areas with potentially plastic strain hazard and the excursions in the plastic field, i.e. keeping the reserves of bearing capacity under control. More than that, by knowing the real efforts, the designer may introduce some "guided" constructive measures, that may reduce the material consumptions without any risk of safety reduction during the service life.

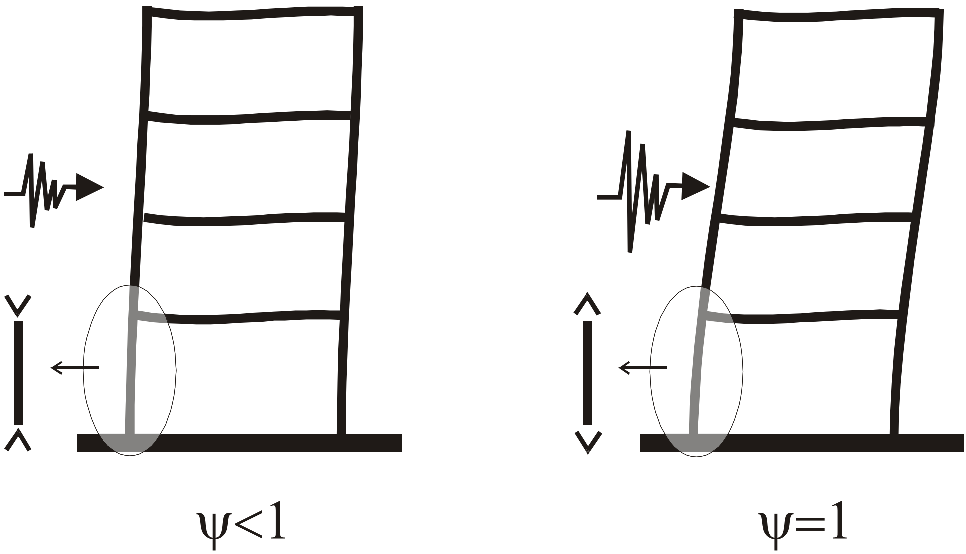

Figure No. 2

4. Conclusions

No matter how correctly and completely they are, the computational models cannot replace the general rules of structural conception. The building that has resistant elements placed in a regular shape that forms an homogenous system with a large degree of static unsettlement provides a better behavior than a structure with inconsistent connections, discontinuities or that allows stress concentrations over a small number of elements, thus enabling the collapse mechanisms. The large degree of static unsettlement allows the stress redistribution over the entire structure, increases the capacity of dissipation of the energy of vibration and the number of the collapse configurations, thus permitting the structure to carry on the real earthquakes. All these elements must become tangible to the designer during the creation process.

A well known fact is that in order to take into account the ductility effect, the effort redistribution, the vibration damping, etc. the actual conventional calculus assumes the reduction of the seismic forces by the means of a single coefficient, ψ. After the seismic computation, the efforts and displacements are increased by multiplication by several coefficients or ratios of capable efforts (i.e. to dimension the columns and shear walls, for the computation of the anchorages and joints, when checking the elements providing stress concentrations or reduced ductility, also in case of stiff horizontal structures, when determining the relative displacements, etc.). In order to avoid these situations, it becomes more intuitive to perform the seismic computation in the elastic domain, with $\psi = 1$, in such way the obtained efforts and displacements become a reference limit, easily interpreted by the structural engineer. In this way one may use differentiated coefficients of reduction for each effort type, element, member end, joint, anchorage, assembly, etc. This reduction can be carried out by multiplication by a subunit coefficient (of ψ nature), or by division by a coefficient greater than one (of ductility coefficient nature). Such a computation will underline the real system behavior and pays attention to the designer about the vulnerable areas. We appreciate this proposal to be necessary when the new aseismic design codes will be written.

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ALGORITHMICAL APPROACH OF BRIDGE CONDITION ASSESSMENT

Rodian SCINTEIE[1]

Abstract

The paper deals with the concept of technical condition of a bridge which represents an evaluation of the entire set of characteristics of the bridge, as technical system, at a certain moment in time. These characteristics combined with the influence of the environment determine the present and future behavior of the system from the point of view of the technical purpose it was constructed for.

In the world of research, different persons or organizations have treated the problem differently. Although there is a common path, finally no common result was obtained. The experience of experts and the research were translated into regulations and even the bridge engineering is the same the results were different. The interpretation and the equations, where they were developed, are generally different. While there is no unique view on this subject the article proposes an algorithm for the technical condition of the bridges based on the Romanian regulations.

The final equation considers that fundamental is the structural condition and the close related items are aggravating factors.

The algorithm is continuing and developing the experience of Romanian bridge inspectors and tries to put the subjective observations on an objective base, offering a framework for quantifying the general condition.

This work is subject of continuous further revision and will be included in the development of the Romanian Bridge Management System.

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1. Introduction

Although a universal notion, technical condition (shortly condition) of a bridge is mainly a concept that is locally defined and accepted by the specialists. It is hard to measure while it depends on many items, facts, and data.

Technical condition of a bridge represents an evaluation of the entire set of characteristics of the bridge, as technical system, at a certain moment in time. These characteristics combined with the influence of the environment determine the present and future conduct of the system from the point of view of the technical purpose it was constructed for.

Characteristic (synonym attribute) indicates either the specific feature proper to a living being, thing, phenomenon which differentiate one entity from another, a being from another etc. or the parameter (a technical data) used to measure the performance of a technical system or of a material.

Obvious such a definition omits or neglects some aspects:

- it is impossible to account for all characteristics of the system;
- it is difficult to evaluate the exact value of the properties, hence approximation is accepted in computation;
- it is impossible to create an absolute exact model of the behavior of the materials used in bridge erection;
- it is impossible to have the precise measure of the action of all external factors on the system.

Consequently, the technical condition of a bridge, as presented before, is a quantitative description approximating the qualitative description of the performance of the bridge as a system in the moment of the evaluation and in the period of time immediately to follow. However, when an agency administrates many bridges it is necessary that it has instruments to assess the condition of each of them in such a way that they may be compared. Also, the possibility that the urgency is quantified must be offered. These two items must warrant a set of actions and they must establish the order of these actions.

2. Forms of bridge condition evaluation

There is no unique view on this subject. Different persons or organizations have treated the problem differently. Although there is a common path finally no common result was obtained. The experience of experts and the research were translated into regulations and even the bridge engineering is the same the results were different. The interpretation and the equations, where they were developed, are generally different.

A common method to describe the technical condition of a bridge is to construct a condition index through aggregation of the individual elements data. Such an index might address larger subsystems as substructure, superstructure, or deck, the bridge as a whole, a number of bridges or the entire network. The level of aggregation depends on the goal aimed and on the scope the interested specialists address.

Different countries, sometimes different agencies from the same country, developed different formulations.

In Romania the bridge technical condition is assessed according to AND 522 regulation – *Instructions for Assessement of Technical Condition of a Highway Bridge*, which was established in 1992 and revised in 1994 and 2002.

This regulation defines 5 quality indices for the material (C_i) and 5 quality indices for the functionality (F_i), as follows:

Table 1. Condition indices according to AND-522

Index	**Name**
C_1	Quality of the main resistance superstructure elements;
C_2	Quality of the superstructure elements that support the deck;
C_3	Quality of the infrastructure elements;
C_4	Quality of the scour, protection elements and access ramps;
C_5	Quality of the deck;
F_1	Adequacy of the traffic;
F_2	Design load rating;
F_3	Age of the bridge;
F_4	Compliance to the design and operation condition;
F_5	Maintenance state.

Corresponding to each preview index, damages, degradations, and dysfunction are identified based on degradation manual and their importance is assessed. Based on gravity each item is evaluated with a number of points that are further deducted from the maximum value of 10. The value of each C_i and F_i is obtained. Finally, a general index is obtained, calculated through summation:

$$I_{ST} = \sum_{i=1}^{5} C_i + \sum_{i=1}^{5} F_i \tag{1}$$

According to the value of I_{ST} a strategy could be chosen. The advantage of such a method is its simplicity. No complex calculation is required. The additive process gains in simplicity but loses in precision.

The fact that the obtained value is not following a smooth increase up to the highest value while the individual indices are increasing makes the formula not so efficient into a prioritization process.

3. The proposed algorithm

It was found necessary to consider the experience of Romanian specialists as a start. The steps already made are important and they are well handled by the factors involved in bridge administration, assessment, and maintenance. A large number of specialists were trained as bridge inspectors and they successfully apply the technique in their day to day activity.

A general image and a consciousness of the bridge technical condition were created upon AND-522 regulation. Its repeated use created an easiness that produces an automatism yielding both an increase in productivity and a better precision of the result due to trial and error process.

The algorithm is based on and it continues the AND-522 regulation with certain amendments. Although it is the specialist's opinion that finally matter, this opinion must be based on observations and numerical evaluation of the encountered failures. The subjective judgment must be objectified.

3.1. The concept of urgency

The experience proves that the higher the degradation and the deduction note the quicker the intervention must be done. This necessity arises no matter the structural subsystem where the degradations occur.

We did not use the indices themselves but the rating of the degradations. Their relation is:

$$DC_i = 10 - C_i \tag{2}$$

and respectively

$$DF_i = 10 - F_i \tag{3}$$

The higher an index the need to action is higher; also the costs to restore a convenient state are higher. This urgency is the same no matter that the damage is found on substructure, superstructure or deck.

All explicit or implicit assumptions of the present proposal refer to real and predicted traffic with no consideration of exceptional or design convoys. This subject must be separately addressed by the administrator.

From the point of view of the urgency, if present traffic did not caused significant damages to warrant intervention it was considered of no consequence that that bridge was designed at an inferior loading capacity. Decision of intervention in such cases is based on different reasons than the condition and may be imposed through different mechanisms than those of the priority. These reasons relate to legislation, politics, international treaties, and military procedures. However, the loading capacity is essential in the decisional algorithm when selecting the action to be done and must chose between (i.e.) bridge rehabilitation and bridge replacement.

Similarly, the age of the structure is not significant in describing the condition. However, this parameter may not be ignored and it is of vital importance when analyzing the bridge life cycle and condition predictions.

3.2. Quality urgency index

The chosen name comes from the fact that the index describes the "illness" condition rather than the "health" condition of the structure. A higher value indicates a poorer condition, hence a higher "urgency" of intervention.

Degradation of the bridge structural elements imposes similar urgency and they might be considered using logical OR operation. Also we may consider DF_1, DC_4, and DF_4 as additive aggravating elements.

All these elements are multiplied with weighting factors in such a way that the value to be within 0 to 100 interval. Based on this judgment the formula is:

$$IU_C = Max(DC_1, DC_2, DC_3, DC_5) \times P_Q + DF_1 \times P_{F1} + DC_4 \times P_{C4} + DF_4 \times P_{F4} \quad (4)$$

where

IU_C quality urgency index;

Max maximum function;

DC_1, DC_2, DC_3, DC_4, DC_5, DF_1, DF_4 degradation indices defined before;

P_Q weight factor for structural damages;

P_{F1} weight factor for lack of adequacy of the traffic;

P_{C4} weight factor for DC_4;

P_{F4} weight factor for DF_4.

Because DC_1, DC_2, DC_3, DC_4, DC_5, DF_1, DF_4 have values between 0 and 10 and we imposed the value for IU_C between 0 and 100, the following values were proposed: P_Q = 6.5; P_{F1} = 2.0; P_{C4} = 0.75; P_{F4} = 0,75.

As a general recommendation, if a bridge is treated separately, a value for IU_C higher than 50 indicates the need for immediate intervention. This doesn't eliminate actions for values lower than 50 when the bridge is considered within a prioritization system.

4. Consequences, observations and conclusions

Using these values we proposed before results in some consequences. Any structural degradation ranked higher than or equal to 8 imposes an immediate intervention even no other indices are considered. Inadequacy to traffic requirements (evaluated through DF_1) may impose intervention even structural degradation is 5.

It is easy to observe that such an equation does not imply derogations and any increase in primary indices would not produce sudden jumps in the urgency index. If an index to describe the health condition then it may be defined as the reverse of the preview index.

$$IS_C = 100 - IU_C \tag{5}$$

Because the condition, as defined before, refers to intrinsic characteristics at a certain moment, extrinsic indices (DF_2, DF_3, DF_5) were not included. However, this does not decrease their importance. They may be used for prediction of condition evolution. Age, maintenance and design load capacity together with traffic and environment are decisive factors for the speed of degradation.

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METHOD FOR COMPUTATION OF PRIORITY ORDER FOR ROMANIAN BRIDGE MANAGEMENT SYSTEM

Rodian SCINTEIE[1]

Abstract

Development of a bridge management system needs an algorithm for the computation of the order of priority. The order is generally based on a composite index. Such an index is necessary and no bridge management system can really perform its tasks without one.

Management of bridges implies a special attention in allocation of the funds to where the technical and economical needs impose and where the benefits are maximal. To establish a correct priority order the index taken into account must have a multi-criterial base.

Administrators, together with research organizations and universities, have created procedures to compute the urgency of intervention using multiple parameters. Selection of these parameters was made starting from the goals observed and the specific conditions concerning the administrator and the roads and bridges network.

This proposal is part of Romanian National Administration of Roads to develop a decision tool for the management of bridges.

The algorithm proposed in the article is very simple and straightforward. It is based on the present regulations and it is not necessary to modify the present way of inspection of the bridges. The equations are very clear, simple and easy to use. Their form is logical and easy to explain.

A unique overall priority index was defined which makes the creation of a list of priority very handy. This priority index includes both degradation and functional influences. Based of simulation, their maximal values were selected in such a way that the final result to be balanced and meaningful. Levels of action were determined and introduced.

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1. Introduction

The work in bridge management implies a special attention in fund allocation to where the technical and economical needs impose and where the benefits are maximal. To establish the priority order on a multi-criterial base, the administrators, together with research organizations and universities, have created procedures to compute the urgency of intervention using multiple parameters. Selection of these parameters was made starting from the goals observed and the specific conditions concerning the administrator and the roads and bridges network.

The methods developed in different countries are not similar due to the subjectivity involved in each one. Specific conditions, different ways to perceive the things, different mentalities lead to different interpretations of the concept of necessity / urgency. Also, the concept of benefit is treated differently or only implicit included in the mechanism of prioritization. Such an example of different vision is the consideration of the influence of the road transportation infrastructure on the environment. While in many countries they don't appear at all among management problems in the Scandinavian countries the environment issues are of a special importance and they surpass many other criteria.

2. Elements used in prioritization

In order to set the order in which the bridges (and in general terms any structures or systems) are treated their technical condition is fundamental. Hence, the condition is the main issue treated and used in all bridge assessment and management systems in the world. Obviously, the concept of technical condition has different meanings and different numerical form expressions for different countries but the principle is the same.

Beside indicators describing the technical condition indicators describing the importance of the structure within the road may be included. Also indices for the importance of the road within the entire road network may be considered. Another important element to bear in mind is the position of the road within the community it serves.

The volume of traffic is an element frequently present in the decision process. The importance of a bridge is higher when the traffic on and under the bridge is higher. Based on this, indices describing the traffic are defined and used in priority definition. Some management systems include among other priority criteria the cost for works. Others include also the results of the benefit / cost analysis. The costs included are both administration costs and user costs. The values for the costs are either explicit expressed or implicit by the use of other parameters that involves differential costs (i.e. detour length). The costs or the benefit / cost analysis results are more and more used as they are required by international financial institutions.

3. Functionality of the bridge

The bridges are structures constructed to establish the continuity of the way where the roads intersects an obstacle. The obstacle might be a deep valley, a river, a railway, or other human creations including another road. The bridge have to be regarded within the network together with the road sector which it is part of, with the obstacle it meets and the community it serves.

In the general context, from different points of view, the bridge might be seen having a certain importance for: the persons involved in traffic, for the inhabitants of the adjacent area, for the industry, and for local trade. The points of view are multiple and are relevant for a category or another.

Based on the importance awarded in different situations one can imagine a computation of the global importance the bridge might have. Assessment of the importance refers to conceiving a numerical computation procedure where the selected functional factor to be quantified. General equation might have different forms, including probabilistic. However, further in this paper we will try to select a minimal number of essential factors and to retain a formula as simple as possible.

3.1 Selected functional factors

Regarded from different points of view, by different persons with different aims and scopes, the bridge may be classified as having certain functionality in correlation with the goal observed. Functional parameters are numerous but in this paper we tried to retain only those considered to be significant from the point of view of the road and bridge administrator.

The fundamental goal of the bridge is to guarantee the continuity and the smoothness of the traffic on a road. The road has certain geometric and traffic characteristics. Also the road has a certain classification within the national network. In the same time the bridge intersects an obstacle which, at its turn, generates an importance. Moreover, the surrounding area can influence the way the bridge importance is perceived. The selected factors are presented in the following table.

Table 1. Factors selected in the evaluation of the functional importance

Index	Significance
UF1	Category of the road the bridge is situated on
UF2	Type of the obstacle
UF3	Index of traffic
UF4	Index of bridge length
UF5	Index of detour length
UF6	Position
UF7	Activity / habitation / circulation under and around the bridge

In our paper we used the preview factors to define the "functional importance index" or "functional priority factor". This index will be further used as an element to establish the priority and the order the bridges will be treated.

Following, we will present the quantification and motivation for the factors.

Category of the road the bridge is carrying

According to present regulations and codes, roads are divided in more categories. For administrators, a road with a higher assigned importance indicates the necessity of a more vigorous intervention on all the adjacent elements, sub-systems and structures.

Road category is not necessarily related to traffic. As an example, the international treaties Romania is party at assign a number of European road corridors. The roads on these corridors have precedence before roads with equal or even higher traffic that are not specified in the treaties.

Starting from these considerations we proposed an order of importance for the bridges on different road categories.

Table 2. Quantifying the road category

UF1	**Category of the carried road**
	Motorway; European road
	Main national road
	Secondary national road; main county road; main streets in municipalities
	Secondary county roads; main streets in towns
	County roads; secondary municipal and town streets
	Streets in communes; others

These values don't vary too much in time and can be determined based on data already available in office as the inspector may find them on the inspection form and he/she only do corrections, where necessary.

Category of the obstacle which the bridge is intersecting

Analogue to the prevue judgment, it is important to consider the type of the obstacle the bridge intersects, whether it is a watercourse, a railway, or another road.

When the bridge is over a road we shall use similar values with the preview factor starting from the idea that the road has similar importance and the damages for traffic interruption are similar whether it is on the bridge or under the bridge. In the same time the cases when the bridge is a passage over the railway were considered.

When the bridge is over a river differences were considered according with of the water stream width. This is assessed by the inspector in the field and is measured at the level of the minor scour. If the water flow is divided in more branches the distance on the bridge axe between the exterior banks of the exterior branches is considered.

The cases where the river is navigable, or there are known seasonal torrents are included.

Table 3. Quantifying the obstacle intersected by the bridge

UF2	Category of intersected obstacle
	Highway; European road; navigable watercourse
	Main national road; railway; water stream larger than 50 meters
	Secondary national roads; main county roads; main streets in municipalities; water stream between 25 and 50 meters width
	Secondary national roads; main streets in towns; water stream between 5 and 25 meters width; valley with abundant seasonal torrents
	Communal roads; secondary streets in towns; water stream narrower than 5 m
	Streets in communes; valleys with no water

If the bridge intersects simultaneously more obstacle the higher note will be taken into consideration.

Traffic influence

Constructed to ensure the continuity of the traffic, the bridge has a higher importance as the traffic on it is higher. For the purpose of this paper the value taken into consideration is the physical traffic, including the vehicle without engine.

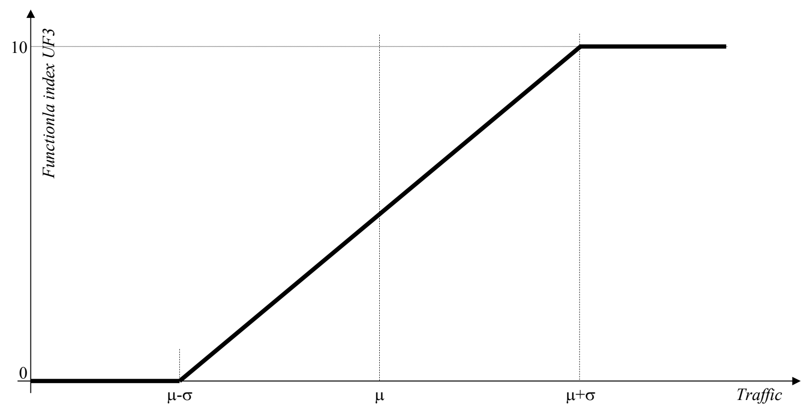

Fig. 1 Function for definition of UF3

For parallel bridges, the concerning part of the traffic will be considered for each one. The average and the standard deviation are considered, at national level or for the part of the network we analyze. Based on them one may define a function for the index UF3. In the preview picture a linear function is presented but other convenient functions are often used.

Length of the bridge

A bridge is considered with much attention when it is longer. A culvert imposes less technical problems while a one kilometer bridge needs a permanent surveillance. Therefore, we set different categories of bridge lengths and an index (UF4) is quantified.

Detour length

A bridge is more important when few possibilities for detour are available. Several kilometers are not very important but when several one must go for more than 10 kilometers when the bridge is closed the bridge must be regarded with permanent care and must be considered important. Some scale for the detour length was proposed so that index UF5 to be quantified.

Position

In the same line of judgment, position of the bridge toward the communities is important. The next table considers different positions:

Table 4. Bridge position

UF6	Category / Position
	Highly industrialized area Historical or special interest bridge
	Urban area
	Mountainous area
	Semi-urban area
	Commune / village
	Rural area

Activities / habitation / circulation

This factor is related to activities in immediate proximity of the bridge or immediately downstream on lower levels. In this category we consider different activities (industrial, agricultural, commercial etc.) or habitations that are located under or immediately downstream of the bridge. Circulation of pedestrians, goods and services are also assessed here. UF7 index is set.

3.2. Functional priority index

Because the factors that influence the importance of a bridge may not be measured with instruments and they are subjective in character we might conclude that there is no absolute criterion for comparison. However, the global importance is higher as it influences and concerns directly or indirectly a higher number of persons, institutions, and businesses. Hence, one may consider that the function describing the importance is cumulative. Each factor presented earlier refers to a class of affected elements. It this way we include the majority of the relevant group of interest.

Subsequently we define the functional priority index as a sum of the preview presented factors:

$$IP_F = \sum_i \left(UF_i \times P_{UFi} \right) \tag{4}$$

Where:

IP_F Functional priority index;

UF_i Functional index *i*;

P_{UFi} Weight factor for the functional index *i*.

In the preview equation the weight factor were set the unitary value ($P_{UFi} = 1$) as the value of the functional indices are already differentiated.

The values of IP_F vary between 0 and 40.

4. Priority order. The proposed method.

Any methodology to set a priority order for action at bridges must consider the technical condition (degradation status) and functionality. Beside this, costs, technology, available resources items may be included.

A bridge may be positioned in a “space of urgency” (see the next picture):

The position in this space indicates simultaneous the two important directions: functional priority and priority due to degradation.

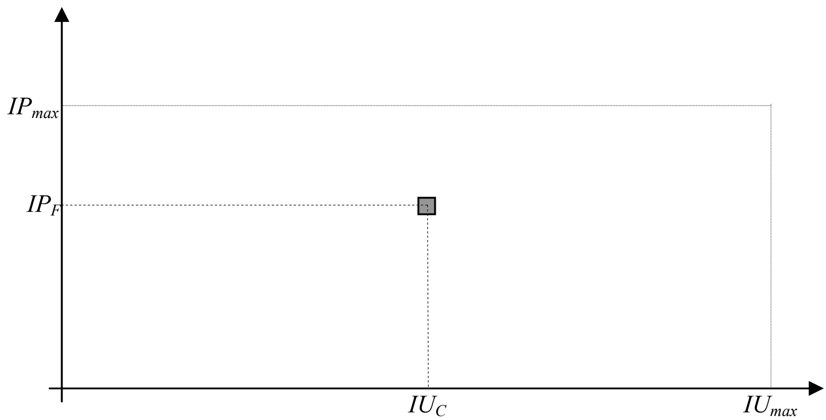

Fig. 2 The space of urgency: Functional / degradation

4.1. Computation of the order index

We have seen that the priority in treating a bridge is given by both quality and functional considerations. The two elements are not cumulative. At extreme values of degradation, the functionality plays no determinant role except for similar values. To combine the two values we consider the distance, in the "space of urgency", from the origin to the position of the bridge.

Mathematically this is expressed by:

$$IO = \sqrt{IU_C^{\ 2} + IP_F^{\ 2}} \tag{4}$$

where:

IO Order index (overall priority);

IU_C Quality urgency index (degradation index);

IP_F Functional priority index.

The overall priority IO takes values between 0 and 107,70329614269(≈108). The value 0 corresponds to minimal urgency and the maximal value correspond to maximal priority in intervention. For eventual esthetical reporting consideration one may force to 100 values of IO larger than 100, but the probability for such situations is very low.

The order index IO helps us to find out where a bridge must be treated faster or it is possible to delay the intervention works. Periodically this index must be reevaluated and new actions are to be selected at the moment.

If no other decision support instrument is used, the actions result from the value of IO. Hence, thresholds may be defined.

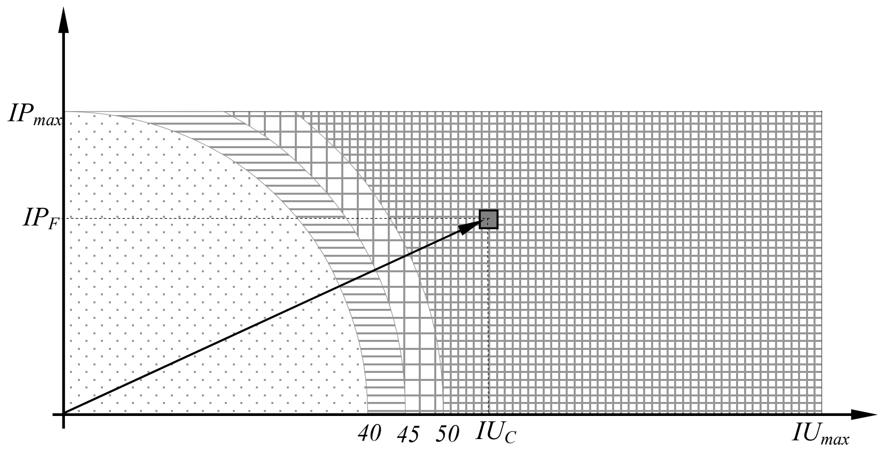

Fig. 3 Level of thresholds

According to the value of overall priority index one may infer and enforce some levels of action. Hereby, such levels are proposed in Table 5.

Table 5. Level of action

IO level	Action
0≤IO<40	Normal maintenance
40≤IO<45	Attention level! periodic follow-up
45≤IO<50	Alert level! Intense survey; intense maintenance
50≤IO	Immediate intervention

These levels impose some degrees in bridge treatment without interdiction for repairing or rehabilitation, as considered appropriate, when the IO index is lower then 50. Decision may be taken according to recommendation of a bridge management system using order index as a priority indicator, based on the available funds and on the benefit / cost analysis.

4.2. Observations to the introduction of order index

Functional priority index induces alone a maximum of 40 when IU_C is 0. Inclusion of a bridge with no physical degradation on the attention level might appear a little bit unnatural. However, let us not forget that, according to the system proposed in this paper, for a bridge to have IP_F at a value of 40 it must simultaneously cumulate the following conditions:

- To be situated in an industrial area or to be an historical bridge;
- To be more than 300m length;
- To be on a motorway or European road;
- To pass over a motorway or European road or a navigation channel;
- To have among the highest traffics in the country;
- Detour length to be more than 50km; and
- Under the bridge or downstream must exist an intense industrial activity.

These conditions are sufficiently restrictive that cannot be met. Such a bridge, if exists, has its own administration and maintenance system.

Because the two indices are asymmetric the functional priority index has the effect of a correction factor. Its influence is higher when the degradation factor is lower. However, considering that extreme values of IU_C are exceptions, when IU_C is in its median zone the influence of the IP_F may not be, in any case, ignored.

4. Conclusions

The algorithm proposed in the article is very simple and straightforward. It is based on the present regulations and it is not necessary to modify the present way of inspection of the bridges.

The equations are very clear, simple and easy to use. Their form is logical and easy to explain.

A unique overall priority index was defined which makes the creation of a list of priority very handy.

The overall priority index includes both degradation and functional influences. Based of simulation, their maximal values were selected in such a way that the final result to be balanced and meaningful. Levels of action were determined and introduced.

Development of such an index is necessary and no bridge management system can really perform its tasks without one. This proposal is part of Romanian National Administration of Roads to develop a decision tool for the management of bridges.

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DEVELOPMENT OF AN ALGORITHM FOR K-SHORTEST PATH

Rodian SCINTEIE[1]

Abstract

The paper presents the theoretical background and a new implementation of k-th shortest path.

The problem of the shortest path is essential in transportation. Its generalization, k-th shortest path, is vital in the field of transportation in studies for solving questions from ITS, road users behavior and dynamic traffic assignment. Even it was well studied so far it was not yet satisfactory solved.

The proposed algorithm has stability and it has low order complexity. For all factor that were considered the dependence is of first order. For a reasonable number of nodes, it can be used on an ordinary computer.

The article includes the pseudo-code description and the high order language computer implementation in such a manner that subject to be clear and easy to understand.

The complexity of the algorithm was established from the theoretical point of view and it was verified by simulation. The simulation results were processed by regression and the dependence found for the running time for different random shape networks respects the theoretical findings.

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1. Introduction

The problem of the shortest path is essential in transportation. Its generalization, k-th shortest path, is vital for solving questions from ITS and dynamic traffic assignment. So far it was well studied but not yet satisfactory solved. It is why the paper proposes the following algorithm.

2. Proposed Loop-Less K-th Shortest Algorithm. Fundamentals

Considering two paths P_1 and P_2 and the ending node of P_1 coincides with the first node in path P_2, we may define the sum of this two paths consisting as a new path obtained by concatenating the links in the paths. We may write that the sum is $P=P_1+P_2$.

In this order, if

$P_1=((N_1N_2), (N_1N_2), (N_2N_3),\dots, (N_{k-1}N_k))$ and

$P_2=((N_kM_1), (M_1M_2), (M_2M_3),\dots, (M_{p-1}N_p))$ than the sum is

$P= P_1+ P_2=((N_1N_2), (N_1N_2), (N_2N_3),\dots, (N_{k-1}N_k), (N_kM_1), (M_1M_2), (M_2M_3),\dots, (M_{p-1}N_p))$. Obviously the length of the sum is sum of the two lengths of the paths, $L(P)=L(P_1)+ L(P_2)$.

A path between two points is minimal if it is not longer than any other path between those two points.

Proposition 1. A path between two points, i.e. O and D, is minimal only if every section of it is minimal. If the minimal path contains the node Q then its sections O-Q and Q-D are minimal.

Proof. Considering $P_{(O-D)}= P_{1(O-Q)}+ P_{1(Q-D)}$ and suppose $P_{(O-D)}$ is minimal but $P_{1(O-Q)}$ is not and exist an other path $P_{2(O-Q)}$ that is minimal. This means $L(P_{2(O-Q)})<L(P_{1(O-Q)})$. In this case there is a new path $P_{2(O-D)}$ as sum $P_{2(O-D)}= P_{2(O-Q)}+ P_{1(Q-D)}$. Its length is:

$$L(P_{2(O-D)})= L(P_{2(O-Q)})+ L(P_{1(Q-D)}) < L(P_{1(O-Q)})+ L(P_{1(Q-D)})< L(P_{(O-Q)})$$

$$L(P_{2(O-D)}) < L(P_{(O-Q)})$$

This means that $P_{(O-Q)}$ is not minimal. The inconsistency is caused by our assumption of existence on a non-minimal section in a minimal path. So the initial proposition is demonstrated.

Proposition 2. To obtain the k-th shortest path from the origin to a destination node is necessary to know not more than the k-th shortest path to subsequent nodes.

Proof. Let us consider $P_{k(O-D)}$ the kth shortest path from origin O to destination D and $P_{k(O-D)}=P_{m(O-Q)}+P_{(Q-D)}$. Also $P_{m(O-Q)}$ is the m-th shortest between origin O and the subsequent node Q and let us suppose $m>k$.

Let now consider paths $Q_{i(O-D)}=P_{i(O-Q)}+P_{(Q-D)}$, where $(i=1,...,m-1)$. Every $P_{i(O-Q)}<=P_{m(O-Q)}$ so $Q_{i(O-D)}<=P_{k(O-D)}$ this means that, in fact, $P_{k(O-D)}$ we considered to be the k-th shortest path between origin and destination has at least other $m-1$ paths shortest than it and it is at least the m-th one $(m>k)$. The contradiction is caused by our assumption of needs of more than k shortest path for subsequent nodes in the k-th shortest path.

The preview proposition is valid when it is permitted to have path with loops. For loop-less path this may happen when the k shortest path to subsequent do not pass through the node considered destination.

3. Development of the algorithm

For solving the k shortest paths problem we propose an implementation of an algorithm that can be generally defined as a Label Setting solving. The main unit is the bubble defined as an entity having the following elements:

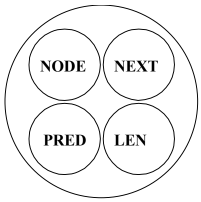

Fig.1 The "bubble"

NODE – the number of the node to be attached to,

PRED – predecessor of this entity in the path,

LEN – the length of the path defined through the predecessor,

NEXT – the next entity attached to the same node but having a higher cost.

Each object attached to a node generates a number of other similar objects according with the number of the links adjacent from the node. If the node already has k objects attached it is blocked. According with Proposition 2 we do not need more then k shortest path to each node. A bubble failing to attach to a node is dissolved. The generated objects are stored within a sorted list and for process we extract only the first one, this means the one with the lowermost cost. Initially all the nodes have no label (no bubbles attached) and the process continues until all nodes become blocked. We present the steps bellow:

> ***STEP0:*** *Define the bubble as an object encapsulating node number (NODE), predecessor (PRED), length of the path from origin to NODE (len), next bubble in chain (NEXT);*
> *Define the main ordered list; Define the auxiliary ordered list. Order the links*

adjacent from each node in ascending length.

Step1: *Set LABEL[i]=NIL, (i=1÷NrOfNodes) (no bubble); Generate a bubble for origin and insert in the sorted list;*

STEP2: *Get from main list the bubble with the lowermost cost. If the NODE in the bubble is already blocked then dissolve the bubble; otherwise attach the bubble to the label of NODE; (the bubble cannot be further removed);*

STEP3: *If the bubble was attached then for each node j where Arc(NODE,j) exists if the node j is not already in path generate a bubble having as predecessor the current one and add it to the end of auxiliary list (the list will be ordered because we keep links in order).*

STEP4: *If any objects in auxiliary list then transfer all bubbles from auxiliary list into the main ordered list.*

STEP5: *If the list is empty then STOP. Otherwise go to STEP2.*

Using this algorithm we may find the k shortest paths from a origin to all the other nodes simultaneously. Once a bubble attached to a node it can not be moved or replaced because we always find and use the lowermost value of the path length. The mechanism is as the bubbles in the sparkling water where a bubble is either dissolved or broken in more bubbles when it touches the node and the generated new objects continue the process.

For each node we may attach only k bubbles and a bubble generates others only if it is attached. This means the total number of bubbles generated in the process is only k*L, where L is the number of links in the network. For the particular case of k=1 we have an algorithm for the shortest path from origin to all nodes in L steps.

Also, because we considered that origin has no meaning to be processed more times, the number of bubbles generated is less then k*L.

4. Implementation of the algorithm

4.1. Data structure

The networks in transportation have a particularity. The number of links for each node is very small comparing to the total number nodes. Trying to describe the links using a node to node matrix would result in a huge matrix with only few cells differing from ∞. We decided to use more matrices that, together, consume less memory space.

We kept the number of links for each node in the variable `NrConects` and in the array `Link` we put the number of the nodes linked with arcs from each node. The number of node corresponds to the index of matrix. Also the cost of each link, the link as defined before, is stored in the same way in the matrix `LinkCost`. To have a better behavior the links and the costs are ordered in ascending value of cost.

```
Type
TConect=array[1..maxNode]of byte;
TLinks =array[1..MaxNode,1..MaxConex]of integer;
TLinkCost =array[1..MaxNode,1..MaxConex] of single;

Var
NrConects:  tConect;
Link:     TLinks;
LinkCost:   tLinkCost;
```

The basic entity, the bubble, is defined as:

```
   pBubble=^TBubble;
   TBubble= Object
      node:integer;
      pred:pBubble;
      len: Single;
      next: pBubble;
      Constructor Init( aNode:integer; aPred:pBubble;
                        aLen: single; aNext: pBubble);
      Destructor Done; virtual;
      Procedure Free;
   end;
```

The bubbles, once generated, are kept in a sorted list. It is the reason we decided to use dynamic memory allocation for solving our problem. In this manner the sorted list is easier to maintain and less time consuming to process. The sorted list has the following structure:

```
pBubbleQueue=^tBubbleQueue;
tBubbleQueue=
      Object
          First, last: pBubble;
   Object
     First, last: pBubble;
     Count: integer;
     Constructor Init;
     Destructor Done; virtual;
     Procedure PutLast(aBubble:pBubble);
     Procedure Append(aBubble:pBubble);
     Procedure InsertInOrder(aBubble:pBubble);
     Procedure InsertAQueue(var aBubbleQueue: TBubbleQueue);
     Function GetFirstBubble:pBubble;
     Function IsEmpty:boolean;
     Function IsNotEmpty:boolean;
     Function BubbleNumber:Integer;
     Procedure Verify;
   End;
```

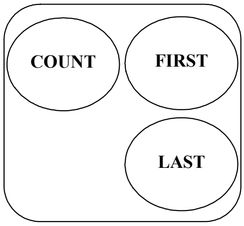

Fig.2 The components of a sorted list holder

For the purpose of our problem, all the bubbles, once accepted, are stored in a double-chained list. This consists in an array which elements are sorted lists. Each list corresponds to a node, and the elements in the list are, in order, the direction for the shortest paths from the origin. Knowing the predecessor in path we can go back to the precedent node until we reach the origin. The array is declared as follows:

```
tNodeLabel= array[1..MaxNode]of pbubbleQueue;
```

In the next picture we may see the way the connections are established on horizontal from the node, and on vertical from each bubble

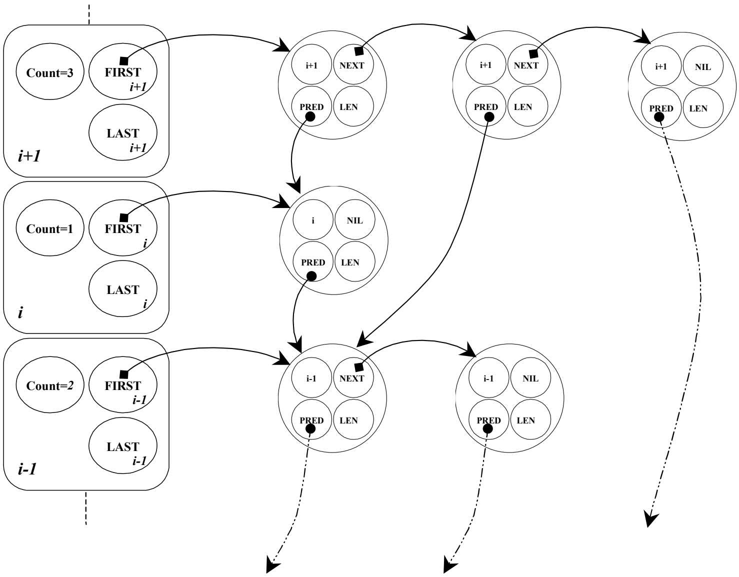

Fig.3 The map of connections

After data is loaded, and we initialize the structures, the program gets each bubble from the queue and processes it until the list is empty.

```
Procedure Process;
begin
     while queue.IsNotEmpty do
        ProcessBubble( queue.GetFirstBubble);
end;
```

Each time the first object in the queue is extracted, it is sent for processing to a routine that actually implements the steps 3 and 4 and partially the step 2.

```
Procedure ProcessBubble( aBubble:pBubble);
   Function IsNotInPath( aBubble:pBubble;
                         Nod:integer): boolean;
  Var cursor:pBubble;
  Begin
    IsNotInPath:=False;
    cursor:=aBubble;
    while (cursor<>Nil) do
          Begin
             if cursor^.node=nod then exit;
             cursor:=cursor^.pred;
          End;
    IsNotInPath:=True;
  End;
Var
  I, nod:integer;
  procedure doInsert;
  Begin
    MQueue.PutLast( new( pBubble,
        Init( link[nod,i],aBubble,
              aBubble^.len+LinkCost[nod,i], nil)));
  End;
Begin
  if aBubble=nil then exit;
  nod:=abubble^.node;
  if (NodeLabel[nod]^.Count<kth) then
    begin
      NodeLabel[nod]^.Append(aBubble);
      for i:=1 to nrconects[nod] do
        begin
          if (link[nod,i]<>_Origin) then
          Begin
            If aBubble^.Pred=Nil
              then doInsert
              else
              if(link[nod,i]<>aBubble^.pred^.node)
                   and
                 IsNotInPath( aBubble,link[nod,i])
                Then doInsert;
          End;
        end;
      Queue.InsertAQueue(mQueue);
    end else begin Dispose(aBubble,Done); end;
end;
```

Running the algorithm, for each node we process it results a number of new bubbles. Trying to insert each one individually in the sorted queue would result in a loose of time by parsing many time the queue. To eliminate this we used an auxiliary queue where we kept temporarily only the objects generated by the processed node. Because we keep the links ordered we have only to put each resulted object at the end of the list. When this is finished we only move at once all the objects from the auxiliary to the main queue. We developed a special subroutine: procedure `InsertAQueue` to do this.

4.2. Time complexity

The algorithm consists in a mechanism that generates objects is a serial way. The number of iterations is established by the network's shape. It is therefore difficult to establish the exact order of complexity. However the worst case is presented in the next picture.

```
Procedure ProcessBubble( aBubble:pBubble);
   Function IsNotInPath(aBubble:pBubble;Nod:integer):boolean;
   Var cursor:pBubble;
   Begin
       IsNotInPath:=False;
       cursor:=aBubble;
       while (cursor<>Nil) do
          Begin
             if cursor^.node=nod then exit;
             cursor:=cursor^.pred;
          end;
       IsNotInPath:=True;
   End;
Var
   I, nod:integer;
   procedure doInsert;
   Begin
      MQueue.PutLast( new( pBubble,
            Init( link[nod,i],aBubble,
                  aBubble^.len+LinkCost[nod,i], nil)));
   end;
Begin
    if aBubble=nil then exit;
    nod:=abubble^.node;
    if (NodeLabel[nod]^.Count<kth) then
       begin
          NodeLabel[nod]^.Append(aBubble);
          for i:=1 to nrconects[nod] do
             begin
                if (link[nod,i]<>_Origin) then
                Begin
                   If aBubble^.Pred=Nil
                      then doInsert
                     else
                    if (link[nod,i]<> aBubble^.pred^.node) and
                         IsNotInPath( aBubble,link[nod,i])
                         Then doInsert;
                End;
             end;
          Queue.InsertAQueue(mQueue);
       end else begin Dispose( aBubble,Done);
                end;
end;
```

<(L-N-1)*k

Number of nodes in path <(N-1)

Number of nodes in the main queue <k*(N-1)*(L-N-1)

Fig.4 Time complexity factors

In the worst case we may consider the complexity is:

C=((L-N-1)*k)*(N-1)+k*(L-N-1)*(N-1)=2*k(N-1)(L-N-1)

Of course the average case complexity will be less than what we get theoretically.

5. Validation of the algorithm

In order to validate our algorithm it was necessary to run a program implementing it on different networks. It was our goal to avoid the influences that particular cases may induce in final results. It is why we decided to run this algorithm on rectangular shape networks with randomly generated costs for arcs.

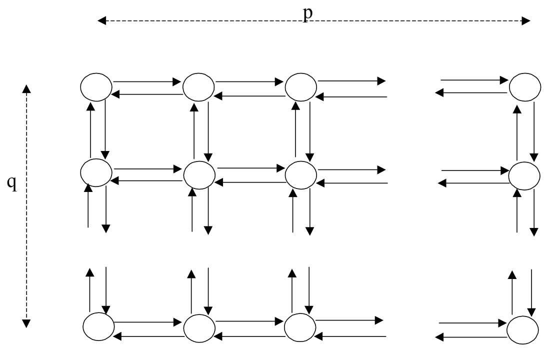

Fig.5 Shape of networks used for tests

The costs of travel on arcs have been generated uniformly distributed so they cover the reality. For each of these network shapes we have generated the more sets of costs. We ran each time the program and we found the average computational time. As an example, in the next picture, we present the dependence of time of the value of k.

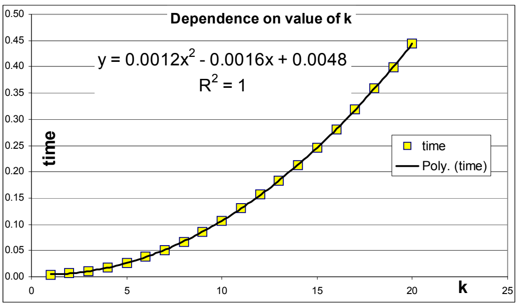

Fig.6 Dependence of time on value of k

We learned that the average computational time depends on the k square. And also we found a good linear dependence on (LINKS-NODES).

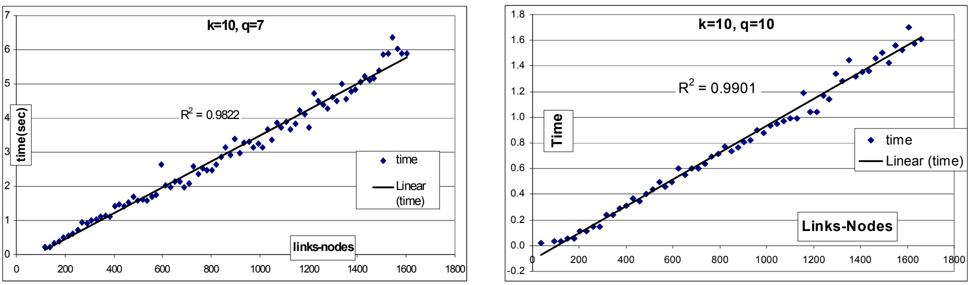

Fig.7 Dependence of time on (links-nodes) for different shapes.

It is very interesting to remark that the computational time varies if the shape of the network varies. For the same number of nodes the average computational time is different for different values of *p* and *q*.

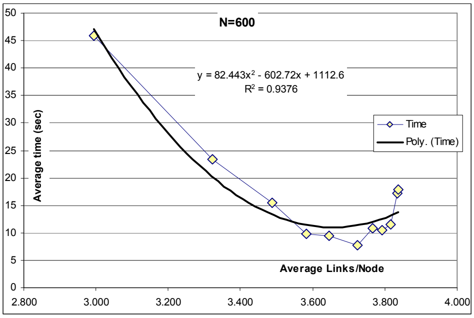

Fig.8 Evolution of running-time on the asymmetric networks

The difference is caused by the fact that for narrow networks, like the one presented in the next picture, the paths for close nodes will contain a higher number of links.

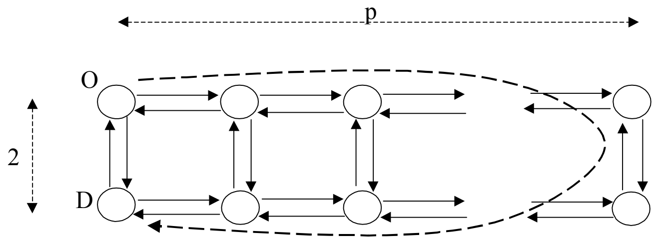

Fig.9 Asymmetric networks

Running the program gave us in certain cases a smaller number of paths than required. This has been proven to happen whenever it is an overloading of certain nodes and links because of unbalanced network (a very small path comparing to the other paths adjacent on the same node). To understand this thing let us look to the following picture (Fig.10). It shows a particular network with some special characteristics.

The result of this algorithm depends on the shape of the network. The software cannot make a difference for this thing. It is useless and meaningless to ask for the second shortest path from node A to B or from node C to F. We may see this, but it is not the purpose of this software to find out this impossibility and the program will try to solve the problem and, of course, it will not. Also, trying to find the first two shortest paths from node A to all other nodes we will find that the first two shortest paths for the nodes D, E, G, and H pass trough node C. Hence, within the logic of our program, using only 2 bubbles for each node the node C will have only one path A-B-C. All the other nodes can not provide a second path without loops.

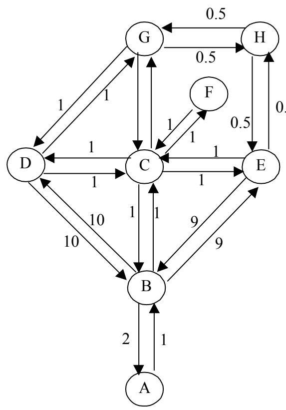

Fig.10 Real world networks

Starting from this finding we tried to avoid the loss of paths, by forcing the paths not to go trough some nodes. However, this will reduce the possibilities of the algorithm only to find the paths from an origin to one specific destination. The restriction is ensuring that no path will be generated for any node to go through destination. So no path with loop will be offered for destination. This also does not guarantee that all the loops will be found. In the network described in the preview figure it may be seen that also no second path will be found for A to B, but also for B to F. In the second case it is impossible to eliminate the overloading of link B-C by blocking node F.

```
begin
    if aBubble=nil then exit;
    nod:= aBubble^.node;

    if (NodeLabel[nod]^.Count<kth) then
       begin
          NodeLabel[nod]^.Append(aBubble);
          if (nod<>Destination) then
          begin
          for i:=1 to nrconects[ nod] do
             begin
                if (link[nod,i]<>_Origin) then
                Begin
                   If aBubble^.Pred=Nil
                      then doInsert
                      else if (link[nod,i]<>aBubble^.pred^.node) and
                              IsNotInPath( aBubble,link[nod,i]) then
                              doInsert;
                End;
             end;
          end;
          Queue.InsertAQueue(mQueue);
       end else begin Dispose( aBubble,Done); end;
end;
```

Restriction for path to stop at destination

Fig.11 Forcing the number of paths for a specific destination

Therefore, it is the duty of network annalist to describe the nodes and links in such manner to avoid in this stage the presence of the isolated node, like A and F. In the first instance the general software will be used to generate paths from origin to as many nodes as possible. For the second step it is necessary to run the Origin-Destination shortest paths finder software to complete the missing paths. This will lead to a consumption of time but balanced by the effectiveness in finding all paths.

6. Conclusions

A new implementation of *k*-th shortest path was proposed. Solving this problem is important in the field of transportation in studies on road user behavior, ITS, dynamic traffic assignment, etc.

The proposed algorithm has stability and it has low order complexity. For a reasonable number of nodes, it can be used on an ordinary computer.

The complexity of the algorithm was established from the theoretical point of view and it was verified by simulation. The simulation results were processed by regression and the dependence found for the running time for different random shape networks respects the theoretical findings.

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SIMILARITY CRITERIA IN THE STRUCTURAL MODELING BY MEANS OF SHAKING TABLES

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Abstract

The choice of similitude scales and of adequate criteria in structural analysis in the case of dynamic loads – especially for investigations on vibrating tables – is presented. An improved criterion taking into account bending modulus ratio was used for some special structures.

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1. Introduction

Structural analysis of buildings erected in seismic areas, must sometimes be completed with experimental investigations, especially in the case of highly redundant structures with non-uniform distribution of bearing elements. According to the development of measuring techniques and testing equipment, physical models are widely used in present time in order to obtain dynamic characteristics, stress–strain distribution, various response parameters, when the prototype is to resist earthquake motions /1/.

The choice of similitude scale values and of adequate criteria in the achievement of physical models is an important task needing a detailed knowledge of testing equipment performances and also a firm resolution about the major factors which are to be taken into account in the study of the dynamic behavior. Some distortions are frequently introduced in the resulting scales in order to obtain sufficiently larges displacements, to inscribe natural frequencies in the range of pulsation of the acting device and to have the possibility of accurately recording experimental data.

In this aim, an improved criterion adequate in the experimental analysis of structures having variable mass and stiffness distribution along the height is introduced. This criterion is based on the bending modulus scale and allows performing investigations in various loading steps. As in order situations, tests in a natural field of gravity are suitable.

2. Similitude laws

Models are usually achieved from plastics, special mixtures having certain mechanical properties and sometimes from the same material as the original. Framed structures have been modeled in order to be tested by means of a set of shaking tables erected on principle of fluid bearings /2/.

The frequency range of the shaking tables and their carrying capacity required that models of special aseismic designed structures to satisfy the corresponding performances of the available testing equipment.

Making use of dynamic similitude laws, two idealized extreme cases can be defined: Cauchy and Froude similitude criteria, that consist in the invariance of the relation:

$$C_a = \left(\frac{F_i}{F_R}\right)_p = \left(\frac{F_i}{F_R}\right)_m = \frac{\rho v^2}{E} \qquad (1)$$

for the Cauchy similitude law, and the invariance of the relation:

$$F_r = \left(\frac{F_i}{F_R}\right)_p = \left(\frac{F_i}{F_R}\right)_m = \frac{v^2}{Lg} \qquad (2)$$

for the Froude similitude criterion.

Starting from Cauchy’s criterion, in the design of two physical models (a steel model and a styplex model) for a thermoelectric power station, similitude scales from table 1, are derived in the situation when acceleration scale:

$$S = \frac{\beta\,\xi}{\mu\,\lambda} \qquad (3)$$

is equal to unity. An experimental check of the separate systems regarding the dynamic characteristics (structure without the upper of the chimney, the chimney alone, the whole structure) was made in order to find a better connection between the different parts of the prototype.

Table 1. Similitude Scales – Usual Criteria

	No.	Name of Scale	Physical Model	
			Plastics	Steel
Arbitrarily Selected Scales	1	Length Dim.	100	50
	2	Young’s Modulus	10	0.143
	3	Specific Mass	2.0	0.306
	4	Poisson’s Coef.	0.85	0.60
	5	Acceleration	1.00	1.00
Derived Scales	6	(Specific) Strain	20	107
	7	Time, Period	45	73
	8	Stress	200	15.30
	9	Total Force	$2x10^6$	$3.83x10^4$
	10	Displacement	$2x10^3$	$5.35\ x10^3$

	11	Velocity	45	73
	12	Damping Force	$2x10^6$	$3.83x10^4$
	13	Damping ratio	$4.48\ x10^4$	53.00

Let be the models described by similitude scales given in table 1. Because acceleration scale is 1, time scale results according to the relation:

$$\theta = \lambda\sqrt{\frac{\mu}{\xi}} \qquad (4)$$

Taking into account that Young's modulus scale ξ and specific mass scale μ are defined by the modeling material, the selected length scale λ defines in the these conditions time scale θ.

A time scale $\theta = 45$ and $\theta = 73$ for the steel model are inadequate in natural period measurements involving higher excitations pulsations than actuating system is able to develop. As can be seen from table 1, the distortion of strains is also very large, ($20 < \beta_y < 70$), which is a disadvantage in strain gage measurements. The displacement scale is:

$$\alpha = \lambda\,\beta_y = \frac{\lambda^2\,\mu}{\xi} \qquad (5)$$

for the styplex model is $\alpha = 2000$ and $\alpha = 3950$ for the steel model.

For example, considering for the model made from plastics a deformation $u_m = 1$mm, for the prototype we have:

$$U_p = \alpha\,U_m = \frac{\lambda^2\,\mu}{\xi}\,U_m \qquad (6)$$

that is a corresponding value of 2.00m.

In the same way, for the model made from mild steel:

$$U_p = 5.350\text{mm} \qquad (7)$$

that is a 5.35m displacement, which is unrealistic for testing requirements. An improved modeling criterion was therefore necessary.

3. Bending modulus criterion

Similitude requirements are applied both for static and dynamic analysis of structures. In order to avoid some of the above described disadvantages resulted from similitude laws, a model analysis based on the bending modulus scale was undertaken. Starting from the ration:

$$\pi = \frac{E_p I_p}{E_m I_m} \qquad (8)$$

selected by trial in order to get adequate displacements for model members and making use of an additional scale of cross–sectional dimensions, we can derive the other similitude scales according to experimental needs. The adopted method is valid especially for flexural elements, since resulting ratios are derived from the expression of the displacements:

$$u_p = \frac{k\, F_p\, l_p^3}{E_p I_p} \qquad (9)$$

$$u_m = \frac{k\, F_m\, l_m^3}{E_m I_m} \qquad (10)$$

where k depends upon the support conditions.

In this way, the scale ξ of the elasticity modulus, the ratio π , above mentioned, the loading scale φ and the acceleration scale S (which interferes in most of derived scales), can be arbitrarily chosen. Investigations of dynamic response using shaking tables will be performed at various loading steps, for adequate frequency range, as was done for some special structures.

Similitude scales are given in table 2 (for a structure without chimney). The derive scales are easily derived in the case of the adopted modeling. For instance there are:

- the displacement scale:

$$\alpha = \frac{U_p}{U_m} = \frac{\varphi\, \lambda^3}{\pi} \qquad (11)$$

- the time scale:

$$\theta = \frac{t_p}{t_m} = \sqrt{\frac{\alpha}{S}} \qquad (12)$$

Table 2. – Similitude Scales – Bending Modulus Criterion

	Scale Name	Notation	Steel Model $\lambda = 50$	
Structure without Chimney	No. of Columns		S_1	S_2
	Specific Strain	β	0.642	0.672
	Normal Stress	γ	0.0918	0.0961
	Sect. Dimensions	λ_y	406	425

Taking into account (11), we can finally write:

$$\theta = \sqrt{\frac{\varphi \lambda^3}{\pi S}} \qquad (13)$$

Specific strain scale is derived from Navier's expression (bending stress):

$$\beta_y = \frac{\varepsilon_p}{\varepsilon_m} = \frac{\varphi \lambda \lambda_y}{\pi} \qquad (14)$$

Results obtained using the described procedure agrees with modal analysis and theoretical computation at seismic actions. Resonant frequencies for the first three vibration modes could be determined.

4. Conclusions

The described modeling procedure has advantageous features in the investigation of physical models by means of available shaking tables. However we stated that using this criterion, the influence of shear strains cannot be simultaneously taken into account. At the same time, since specific weight of the structural members is included in the arbitrarily chosen loading scale φ, the determination of stress distribution for a given acting program must be achieved for a certain loading step corresponding to the actual value. Having in mind the restrictions required in the use of the described criterion, the procedure is adequate in many similar cases in the experimental stress analysis of dynamically acted constructions.

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SOME REQUIREMENTS IN THE MODEL ANALYSIS OF STEEL STRUCTURES

Violeta–Elena CHITAN[1], **Lucian STRAT**[2]

Abstract

The use of physical models in the analysis of mold steel frames behavior, besides some well–known advantages, presents also the reliability of permitting the achievement of sufficiently small samples made of the same material. In the experimental study of steel structures subjected to dynamic and earthquake–like actions, we do resort in most cases to model investigations.

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1. Introduction

In this paper are briefly described some requirements of the dynamic similitude laws when modeling in a natural field of gravity and when using the same material for the model as for the prototype. We also present some results obtained in the investigation of a three-storied steel structure having two spans. The model of this structure was tested on the fifteen tons shaking table in order to get the dynamic characteristics in different stages of behavior.

2. Similitude Laws

The dimensionless parameters interfering in the experimental study of the physical phenomena are related by criterial equations of the form:

$$\varphi(\pi_1, \pi_2, \ldots \pi_{n-h}) = 0 \qquad (1)$$

where: $\pi_1, \pi_2, \ldots \pi_{n-h}$ are dimensionless complex quantities which are to be experimentally determined, *n* is the number of physical quantities which are taken into account, *k* is the number of fundamental quantities selected from the Scale of the International System Unities or free of it. This number is equal to 3 when certain distorsions are not interfering.

In the theory of similitude the dimensionless parameters are denoted similitude criteria and they suppose the same numeric values for the model and for the original. So, whether from the point of view of inertial forces F_{ip} and F_{im} the ratio:

$$\frac{F_{ip}}{F_{im}} = \frac{m_p a_p}{m_m a_m} = \text{const.} \quad (2)$$

is kept of the same value and whether we denote:

$$\mu = \frac{m_m}{m_p}, \quad \alpha = \frac{l_m}{l_p}, \quad \xi = \frac{t_m}{t_p} \qquad (3)$$

we obtain the expression of Newton's criterion:

$$\frac{F_p}{F_m} = \frac{\xi^2}{\alpha\,\mu} = M_e \qquad \text{or} \qquad \alpha = \frac{\xi^2}{\mu\, M_e} \qquad (4)$$

In the design of physic models, one most also use results following from applying Cauchy's criterion which express the ratio of the inertial loads to the elastic ones. In this case we have:

$$\frac{F_i}{F_e} = \frac{\rho\, l^3\, a}{\sigma\, l^2} = C_a \qquad (5)$$

Let us transcript (5) for the original and for the model:

$$\frac{\rho_p\, l_p^3\, a_p}{\sigma_p\, l_p^2} = \frac{\rho_m\, l_m^3\, a_m}{\sigma_m\, l_m^2} \qquad (6)$$

therefore:

$$\frac{l_p}{l_m} \cdot \frac{a_p}{a_m} \cdot \frac{\sigma_p}{\sigma_m} \cdot \frac{\rho_m}{\rho_p} = 1 \qquad \text{or} \qquad \alpha\, s \frac{1}{\beta} \delta = 1 \qquad (7)$$

Table 1 – Symbols of Similitude Scales

No.	Name of Scale	Symbol of scale	Model vs. prototype	Relations between Scales	Numerical Value
1	Gravity Acceleration	s	a_m/a_p	s	1
2	Linear Dimensions	α	l_m/l_p	α	1/3
3	Youngh's Modulus	ρ	E_m/E_p		1
4	Specific Mass	δ	ρ_m/ρ_p	δ	1
5	Specific Strain	γ	$\varepsilon_m/\varepsilon_p$	$\gamma = \alpha \cdot \delta$	1/3
6	Unitary Stress	β	σ_m/σ_p	$\beta = \gamma \cdot \rho$	1/3
7	Time, Periods	ξ	t_m/t_p	$\xi = \sqrt{\alpha\gamma}$	1/3
8	Lumped Force	φ	P_m/P_p	$\varphi = \alpha^3 \cdot \delta$	1/27
9	Bending Moment	M	M_m/M_p	$M = \alpha^4 \cdot \delta$	1/81
10	Bending Modulus	π_{EI}	$(EI)_m/(EI)_p$	$\pi_{EI} = \alpha^4 \cdot \rho$	1/81

We can deduce from (7) the value of gravity acceleration which must be imparted to structural model in order to reach a given a_p for the full scale frame (according to a certain degree of earthquake motion):

$$a_m = \frac{\beta}{\alpha\,\delta} a_p \qquad (8)$$

if $a_m = a_p$, we can write:

$$\beta = \alpha \cdot \delta \qquad (9)$$

Since according to Hooke's law, we admit $\beta = \rho \cdot \gamma$ and models are carried out from prototype material $\rho = 1$ it follows $\beta = \gamma = \alpha \cdot \delta$. Taking also into account that $\delta = 1$, in this case we have:

$$\beta = \gamma = \alpha \qquad (10)$$

Therefore, in view of performing model tests at a length scale $\alpha \neq 1$ in the situation $a_m = a_p$, it is necessary to adopt a distortion for the specific strain scale γ which, according to (9), becomes of the same value as α and β.

It was proved to be suitable for the testing of any structural models on the fifteen ton shaking table to use similitude laws for the situation when the acceleration scale is unitary. This enables to avoid centrifugation procedures or the additional loading mounted in order to compensate the gravity weight.

The models were made employing mild steel rectangular bars scaled to $\alpha = \gamma = 1/3$. Since $\gamma \neq 1$, a distortion for the specific strain scale was introduced; the measured values are adequate for the accuracy of the available strain gage equipment. In this case, the requirement regarding the Poisson's scale of the model and prototype, which must be unitary, is certainly fulfilled.

As can be seen in fig. 1, the design of the metallic structural model (having two spans and three levels) scaled to α = 1/3, required also the distortion of the geometrical shape of prototype section. The prototype elements are achieved from I shapes (fig. 1a) and the model elements are made from equivalent rectangular bars (fig. 1b). The tested frames are system with nine degrees of freedom.

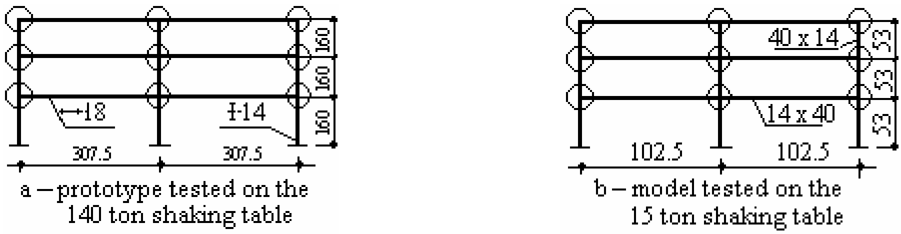

Fig.1–Three Storied Steel Structure

The models tested on the fifteen ton shaking table are designed according to the resulted bending modulus scale.

The live load of the fifteen ton shaking table comprises 10 tons and the frequency domain varies from 1 to 16 cps. The resulting scales agree with the performances of testing equipment.

The relations between the adopted similitude scales are presented in Table 1. We can state that the time (period) scale:

$$\xi = \sqrt{\alpha\gamma} \qquad (11)$$

resulted equal to 1/3; this enabled to do resonance investigations for the first and the second mode of vibration.

In order to benefit by the plastic compliance reserves of elements achieved from I shapes, the columns of the structure were disposed after the minor axis; so were the similar rectangular bars which formed the columns of the model. In this way appearance of plastic hinges was directed in the columns, the fully plastic moment of girders being greater as the columns one. The columns (I 18 shapes) have been modeled by 40 x 14mm equivalent rectangular columns sections, disposed after the minor axis. The girders (I 14 shapes disposed after the major axis) have been modeled by 40 x 14mm bars disposed after the major axis.

3. Some Results in Modeling Steel Structures

In the specialized Research Stations, devoted to dynamic and earthquake – like investigations, as for instance, the Experimental Institute for Structural Models from Bergamo /2/, structural models at different scales and types are usually tested, in order to give a better choice for their aseismical design. By means of the shaking tables from Iasi Seismic Station, a set of investigations on various steel structural models were also carried out. We shall briefly refer to the above mentioned model tested on the fifteen ton vibration table, achieved in view of determination of dynamic characteristics in the elastic and postelastic stage of behavior.

The lumped mass in different steps was achieved by means of nine transverse boxes located between the steel frames. The load transmission to the joints of the twin frames was made by interesting a device that enables free rotations both on the longitudinal and on the transversal direction.

The experimental results concerning the natural frequency and the damping values are given in table 2. We note that the recorded values in step II correspond to the appearance of plastic zones in the bottom of the first level columns and the values in loading step III to the forming of the mechanism of collapse; in step III, following the alternate plasticization at dynamic repeated cycles, marked plastic zones appeared (plastic hinges) at both ends of the columns of the first story.

Table 2 – The experimental results concerning the natural frequency and the damping values

Loading Step	Natural frequency cps		Damping %		Residual Deflection mm		
					Story		
	Elastic	Postelastic	Elastic	Postelastic	I	II	III
Step I	3.96	-	0.703	-	1.9	2.0	1.9
Step II	2.59	2.52	1.080	1.14	3.1	3.3	22.8
Step III	2.04	1.85	1.230	1.66	15.4	14.1	14.0

In the last stage, natural frequency decreased by 10% about and damping increased by 35%. Before failure occurred, the horizontal residual set for this structure was roughly 15mm. The recorded values at each story of the model are in the table 2. In case of the tested structure, the analysis of predicted sideway mechanism of collapse yields for static equivalent loads a lower bound of the collapse load, whose dimensionless value is given by:

$$p_2 = \frac{P_2 h}{M_p} = 2 \quad (12)$$

This lower bound was obtained applying the principle of virtual work. In fact, collapse took place for an equivalent dynamic load larger by 20% as the value computed with the expression (11) that means for tests carried out in step III. The shape of the actual mechanism of collapse concured with the computed one. This represents an interesting conclusion permitting to maintain that it is a similarity between plastic computational procedures for static loads and the same procedures in the case of dynamic actions, in certain conditions.

Further investigations concerning the dynamic and earthquake – like response of structural models in elastic and postelastic stages will be carried out, taking into account that similitude requirements must be observed.

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