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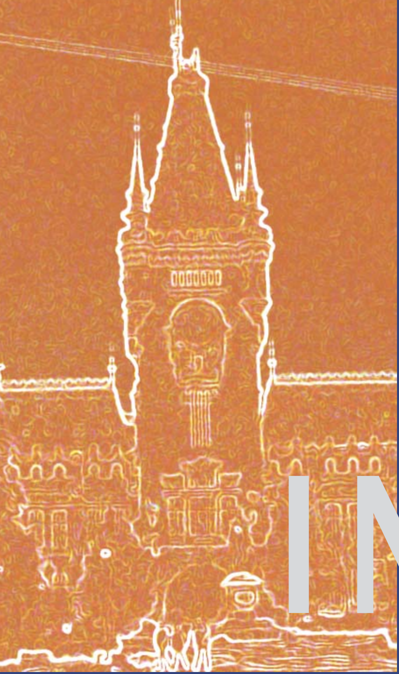

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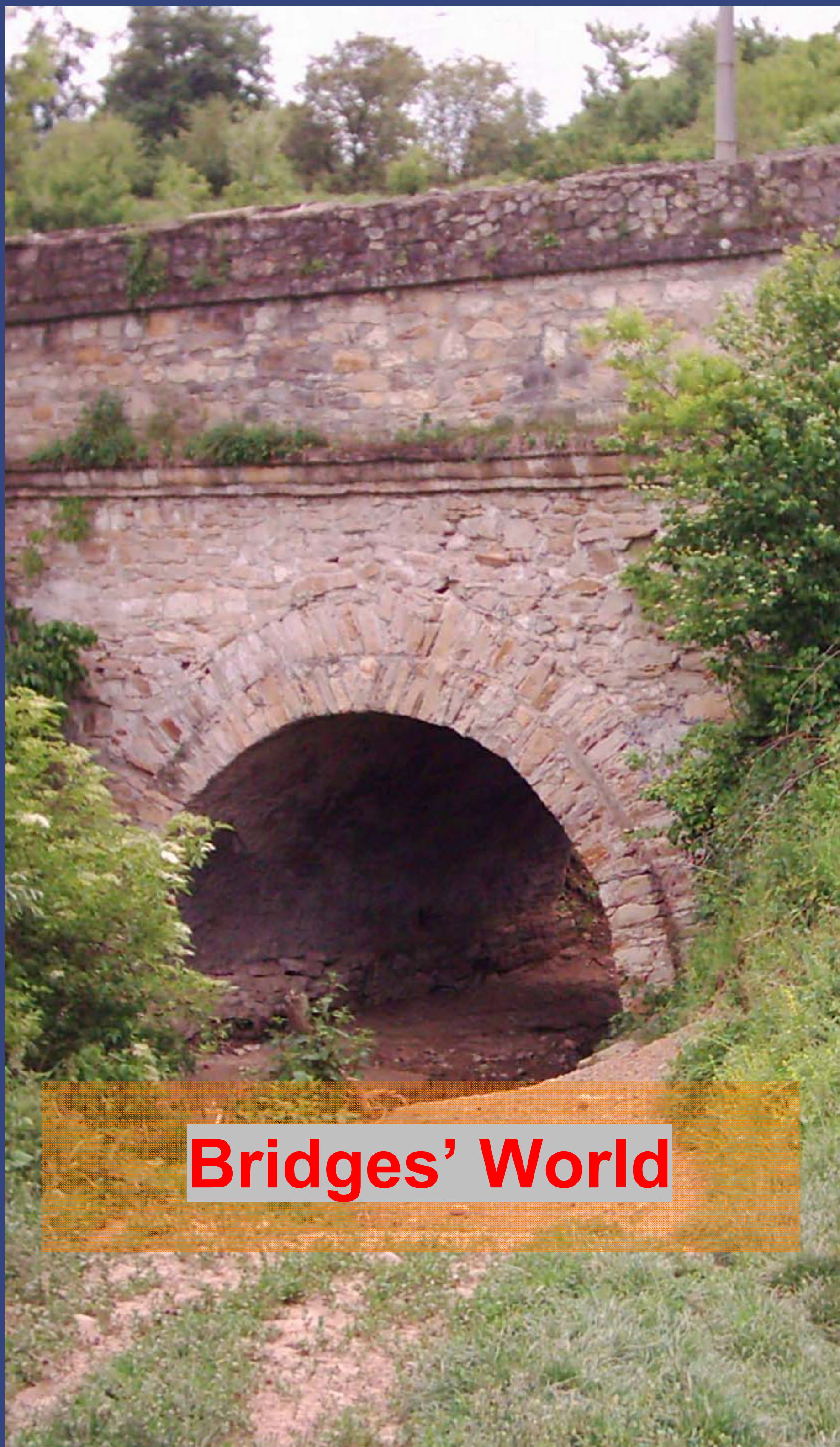

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Decentralized seismic response control of a long span cable-stayed bridge for a benchmark problem

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Abstract

Complexity is a central problem in modern system theory and practice. The ability to study large complex systems is greatly enhanced by modern computing machinery. A theory of large-scale complex control systems is rapidly developing, supplying powerful tools that enable to solve effectively more and more practical problems in different areas.

Potential motivating advantages for using decentralized control schemes are in the reduction of transmission costs within the feedback loop, in the increasing of the reliability of the control operation in case of sensor/actuator/controller failures, the reduction of overall computational effort and the ability of parallel implementation in real time.

It is well known that the control of flexible structures represents a new, difficult and unique problem, with many difficulties in the processes of modeling, control design and implementation.

This investigation presents an overlapping decentralized control design for a cable-stayed bridge benchmark which was proposed within the structural control community to design and compare control schemes. The cable-stayed bridge has two towers as main structural elements. This naturally suggests the overlapping decomposition of a finite element overall dynamic model into two subsystems sharing a common part. Each subsystem is formed by a tower, adjacent cables and a part of the deck. The common shared part is formed by the central part of the deck.

The paper firstly describes the problem and the objectives of the control. Then the overlapping solution is proposed and the corresponding algorithm is shown.

The idea of decentralization of control has been numerically tested using a SIMULINK scheme and compared to the benchmark sample centralized control design using the LQG design.The performance of the overlapping decentralized control design has been assessed by means of given benchmark evaluation criteria, eigenvalue analysis and time responses. The dynamics of the closed-loop

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benchmark model with the overlapping local controllers exhibits an acceptable behavior though slightly worse than in the centralized case.

1. INTRODUCTION

Benchmark structural models have been proposed in recent years as challenging problems to the structural control community to design and compare control schemes for buildings and cable-stayed bridges subjected to seismic and wind excitations [1].

On the other hand, a theory of large-scale complex control systems is rapidly developing, supplying powerful tools that enable to solve effectively more and more practical problems in different areas. Particularly, the emphasis is laid on a theory synthesizing control laws under decentralized information structure constraints [2].

Overlapping decompositions and decentralized control schemes have been applied in different systems as buildings [3,4], bridges [5,6], car suspensions [7], telescopes [8], longitudinal motion control of a platoon of vehicles [9,10], etc.

In this paper, it is attempted to explore the possibility of applying overlapping decentralized control tools to the cable-stayed bridge benchmark control problem proposed in [1]. This problem deals with a long span cable-stayed bridge with two main towers, each one with over hundred cables attached to.

Among the wide variety of control methods available for decentralized control design, the overlapping decentralized LQG design with an infinite time horizon is adopted. Also, he expansion-contraction concept of extension has been employed. The sample LQG design in [1] has been selected as a reference case. Simultaneously, the control strategy implementation constraints and procedures required in [1] are a-priori satisfied when considering the overlapping decentralized LQG design. Further, the extension ensures contractibility of overlapping controllers [11].

The paper constructively describes a procedure in which the overall finite element model (FEM) of the benchmark cable-stayed bridge is decomposed into two overlapping subsystems. By expanding the original LQG problem into a larger space, the overlapping information sets become disjoint and the expanded LQG problem can be solved by standard decentralized methods. This design is made by performing a model reduction for each subsystem in expanded space. In this study the effectiveness of the overlapping decentralized control approach is tested by numerical simulations. To measure the performance, closed-loop eigenvalue analysis, calculation of evaluation criteria given in the benchmark problem, and

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analysis of time history response for selected earthquake excitations are used. The paper is an extention of the results presented in [12].

2. PROBLEM STATEMENT

Consider the cable-stayed bridge illustrated in Figure 1. It is composed of two towers and 128 cables. The bridge is excited by an earthquake longitudinal acceleration. Five accelerometers and four displacement sensors are used to supply feedback information for the control, which is produced by 24 hydraulic actuators located between the deck and the towers and the end supports acting to apply longitudinal forces on the deck. A complete physical description of the bridge, a finite element model and a MATLAB/SIMULINK simulation framework are given in [1] as a benchmark for control design. A centralized LQG control design is also presented in [13].

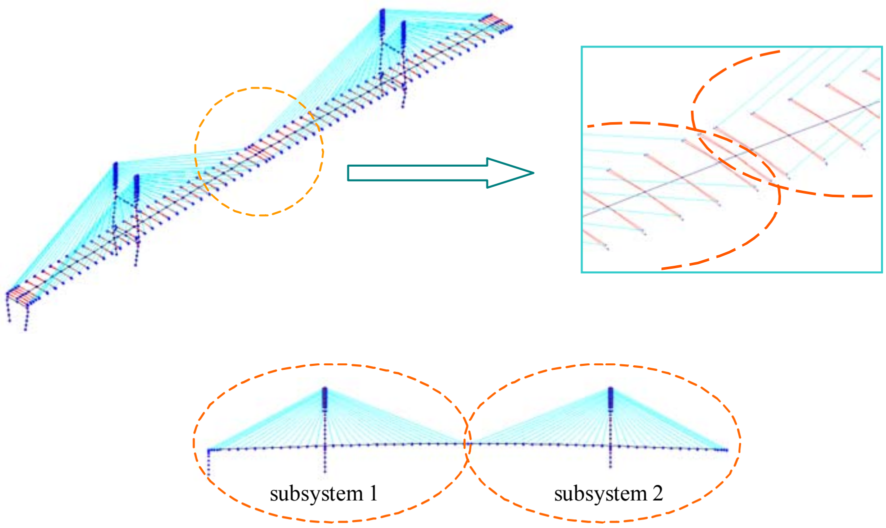

Figure 1. Bridge model and its overlapping decomposition structure

The objectives of this study are the following:

1. To propose a convenient overlapping decomposition of the bridge structure with overlapping subsystems.
2. To design an overlapping decentralized LQG active control strategy.
3. To perform simulations to assess the dynamic behavior of the benchmark bridge model when using the implemented decentralized control.

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4. To assess the performance of the overlapping decentralized control by checking eigenvalues for the closed-loop system, calculating benchmark evaluation criteria and analyzing dynamic responses under selected benchmark earthquake excitations in comparison with results obtained with the sample centralized control design.

3. SOLUTION

This section is divided into three parts: Overlapping decomposition, Overlapping decentralized control design and Simulation results

Overlapping decomposition

The decomposition of the bridge model into two subsystems is proposed. Each subsystem corresponds to one of the towers, the cables attached to it and the part of the deck where the cables are attached. This is illustrated in Figure 1. Both subsystems are interconnected through the center of the bridge. It corresponds with the part of the deck where no cable is attached. The overlapping decomposition procedure considers the towers strongly connected via deck in the part where no cable is attached. When the original model is extended, it defines a state space model in a larger space with the structure of disjoint subsystems and interconnections.

More precisely, the overall original FEM model consists of 838 states. By properly re-arranging the components of the state, input and output vectors, the overall model can be split into two overlapping subsystems. These subsystems have 414 and 434 states. The overlapping subsystem has 10 states. The overlapping common part includes one sensor but no actuator.

Overlapping decentralized control design

First, the benchmark sample LQG design has been selected as a reference. Further, the control subsystems have been defined with the same locations and models of sensors and actuators as in the reference case. The global model has 8 control inputs and 13 measured outputs. The expanded system has two subsystems with 414 and 434 states, 4 and 4 control inputs, and 7 and 7 measured outputs, respectively. There are 14 measured outputs in the expanded space because the overlapped part includes a sensor that is also expanded, i.e. doubled in the expanded space.

A decentralized control law is proposed for each free subsystem by combining its model reduction and the LQG design on the reduced order subsystems. Model reduction first forms a balanced realization and then condenses out the states with relatively small controllability and observability grammians. An algorithm follows:

Algorithm

1. Expand the original LQG problem with identified overlapping subsystems into a larger expanded space.
2. Perform model reduction for each subsystem. Select a minimal order of the subsystem's states ensuring the stability of the reduced-order models.
3. Perform the LQG with preselected weighting matrices for reduced order subsystems.
4. Contract and implement the local controllers into the original overall FEM model and run simulations.
5. Evaluate the results by computing the given benchmark evaluation criteria, the closed-loop system eigenvalues and the dynamic responses, all in comparison to the centralized control design reference case.
6. Tune the control laws by repeating the simulations for different weighting matrices until acceptable results are reached.

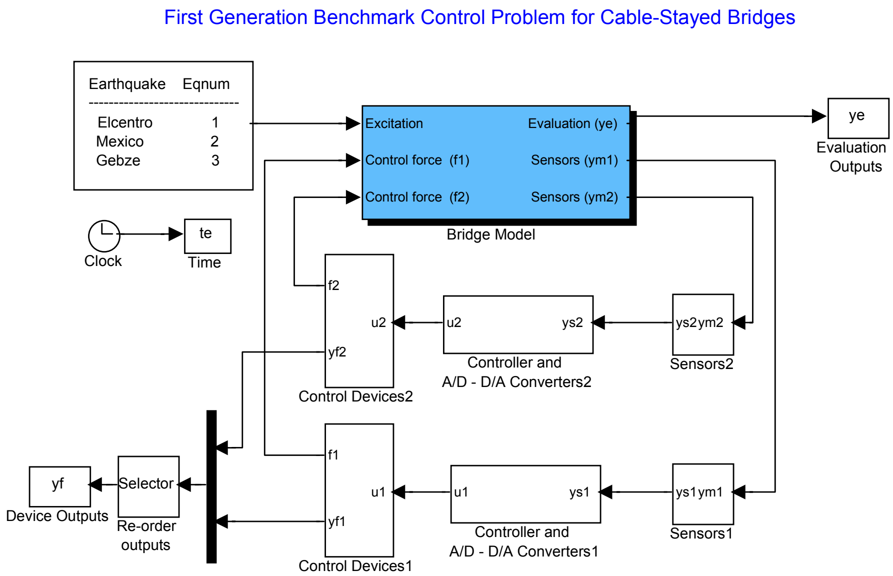

Figure 2. SIMULINK diagram of the decentralized control scheme

The expansion/contraction process with extension, the model reduction and the LQG design are performed using well-known algorithms. MATLAB/SIMULINK and Control System Toolbox are used to help in this design and also to perform the numerical evaluations. Figure 2 shows the SIMULINK diagram with the two overlapping decentralized controllers. Overlapping does not appear in the resulting controller because there is no actuator in the overlapped part. However, a common part of both subsystems formed by a part of the deck is actuated twice.

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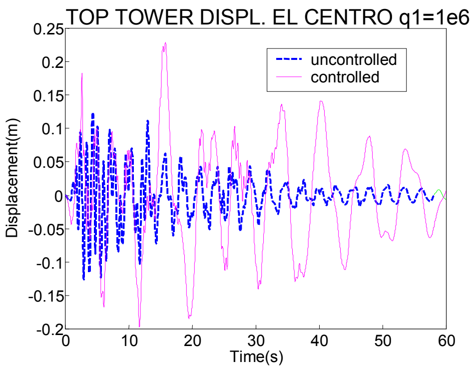

Figure 3: Tower top displacements

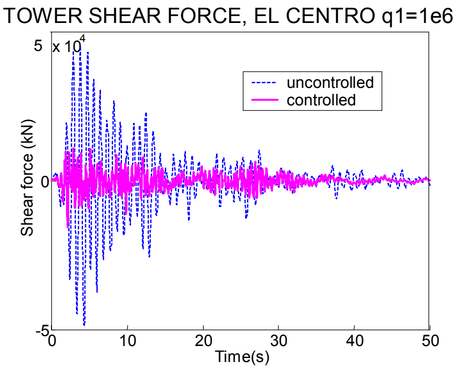

Figure 4: Tower base shear forces

Simulation results

For the overlapping decomposition of the FEM model described above, the model reduction results in reduced-order stable subsystems with a minimal dimension of 34 states for each subsystem. The decentralized LQG control design has been performed with the weighting on the state defined by the identity matrix multiplied by a scalar q_1. Some results are summarized in the following.

For one of the towers, Figures 3 and 4 show the top displacement and the base shear force, respectively, for q_1=1e6 in comparison with the uncontrolled case. The excitation is the acceleration of El Centro earthquake. From Figures 3 and 4, it is observed that the improvement achieved in reducing the tower base shear force is obtained in exchange for an increase (within an acceptable range) in the top tower displacement response with respect to the case without control.

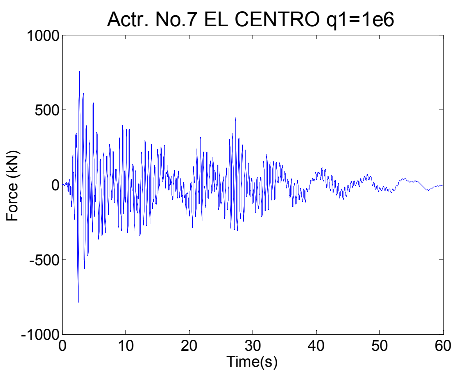

Figure 5: Actuator control force

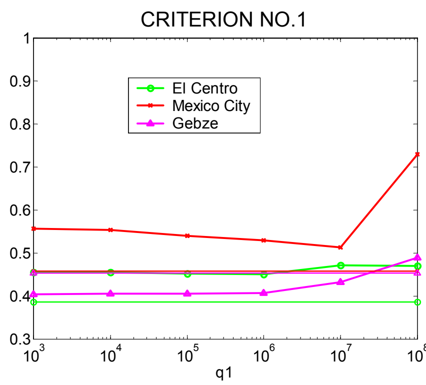

Figure 6: Benchmark criterion No.1

Figure 5 displays the control force supplied by one of the actuators located at the connection between the tower and the deck. Figure 5 shows that the control force is

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acceptable since it will always remains without reaching the saturation value of 1000 kN defined as a maximum in the benchmark sample application.

Table 1: Eigenvalues comparison (Hz)

Freq. no.	Open loop	Closed loop centralized	Closed loop overlapping
1	0.1619	0.1619	0.1619
2	0.2667	0.2667	0.2667
3	0.3725	0.2682	0.3725
4	0.4547	0.3617	0.3852
5	0.5017	0.3724	0.3877
6	0.5652	0.4547	0.4547
7	0.6190	0.5017	0.4864
8	0.6489	0.5653	0.5017
9	0.6968	0.6190	0.5054
10	0.7097	0.6489	0.5653

In order to compare the results obtained with the overlapping decentralized control with those given in the centralized reference case [1], Table 1 gives the first ten eigenvalues of the closed-loop system modes. The overlapping decentralized case corresponds to q_1=1e6.

The first two modes remain unchanged by the proposed feedback controllers. The third mode is changed in the centralized sample control design case, but it remains unchanged in the case of overlapping decentralized controllers. The other modes are in close range. This may be interpreted as a slightly better performance of the centralized control as compared to the overlapping decentralized control case.

The benchmark evaluation criterion No. 1 is presented in Figures 6 for the overlapping decentralized LQG control design with varying scalar q_1. Direct comparison with the benchmark sample centralized LQG control design case given by Table 4 in [1] is included by horizontal lines in this graph. This comparison is made for the three different earthquakes provided by the benchmark. The thick horizontal lines show the nominal (uncontrolled) values.

Criterion No. 1 is a ratio between the maximum absolute shear force at the bridge tower base over the same for the situation without control.

From Figure 6, it is observed that the overlapping decentralized control acted worse than the centralized control in the cases of earthquakes Mexico and El Centro and better in the case of Gebze Earthquake. Also, from the same Figure 6, it

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is very useful to identify an "optimal" value for the parameter q_1, in this case 1e6 has been selected.

4. CONCLUSIONS

The paper has presented simulation results of the overlapping decentralized LQG design for the cable-stayed bridge benchmark performed in a SIMULINK/MATLAB scheme. Two overlapping subsystems are considered, where each subsystem is composed of a tower, part of the deck and the set of corresponding attached cables.

The overlapping decentralized model has used the same locations and models for sensors and actuators as the reference (benchmark) case. Then the original model is expanded into a larger state space model with disjoint subsystem-interconnection structure by using the notion of extension. The proper decentralized design starts with free subsystems model reduction in expanded space. It includes also the expansion of a sensor appearing in the overlapped part. The reduced order subsystems are used as control design models.

The results look promising and confirm expectations. They are slightly worse than in the case of the sample centralized case but lie within acceptable ranges. They satisfy also the requirements on cable tensions that are known a-posteriori. This encourages applying other overlapping decentralized control design methods to this problem in the future.

Acknowledgements

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Considerations on defectation condition assessment

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Summary

Assessment of bridge condition implies a great responsibility both from the personnel involved in performing the operations and from the administrator of the bridges. Importance is given by the fact that based on the current condition as an overall index and on the observed degradations one may establish the works program indicating the afferent emergencies and costs.

In the assessment of the technical condition of the bridges in Romania a study has been conducted. This study involved specialists from CESTRIN and from GETEC Company from France. In a cooperation partnership jointly financed by Romanian National Administration of Roads (NAR) and French Minister of Finance, more than 1000 bridge from the national roads network were inspected. The project was carried out in the European Union pre-accession programs and it encloses the structures situated on the two pan-European corridors, which are covered by TINA (Transport Infrastructure Needs Assessment).

The two methods presented hereby have common characteristic but also differences. They both respect the general criteria for condition assessment by visual inspection but they differ by complexity and detail level.

In the method developed by GETEC, the inspection of a bridge has more objectives: knowing the assets and completion of existing databases; assessment of structural condition and the cost required for its restoration; determining the priority and emergency for sorting the maintenance and rehabilitation interventions.

KEYWORDS: bridges, administrator, assessment methods, inspection, degradations, strategy, maintenance, reparation, rehabilitation.

1. INTRODUCTION

Assessment of bridge condition implies a great responsibility both from the personnel involved in performing the operations and from the administrator of the bridges. Importance is given by the fact that based on the current condition as an overall index and on the observed degradations one may establish the works program indicating the afferent emergencies and costs.

While the assessment methods are more detailed and based on exact measurement the evaluation of the condition is more correct.

Administrator's responsibility consists in selecting those assessment methods which better describe actual situation and those evaluators that master the knowledge required in application in practice of the selected methods.

In the assessment of the technical condition of the bridges in Romania a study has been conducted. This study involved specialists from CESTRIN and from GETEC Company from France. In a cooperation partnership jointly financed by Romanian National Administration of Roads (NAR) and French Minister of Finance, more than 1000 bridge from the national roads network were inspected. The project was carried out in the European Union pre-accession programs and it encloses the structures situated on the two pan-European corridors, which are covered by TINA (Transport Infrastructure Needs Assessment).

2. BRIDGE ASSESSMENT METHODS

The study consisted in field inspections with mixed teams and technical condition evaluation using two methods: the one currently in use in Romania and a method developed by experts from GETEC with respect to the regulations from France. These methods are similar because they are both based on visual inspection and on the experience of the inspector. They differ by the level of detailing involved and the accent they put on different elements.

2.1. Romanian method

Bridge inspection in Romania is conducted with respect to AND522-2002 and the Degradation Manual [2]. No supplementary manuals or guides were drafted so far. Even efforts were made, the works to an "Inspector's Manual" were stopped due to lack of financing.

In this method, inspection implies filling-in an inspection form with structures' important data and degradations as they are identified by inspector. Degradations are grouped by main sub-systems of the structure.

An important chapter is given to the functionality.

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According to the regulation AND522/2002 (*Instructions for bridge condition assessment*), five quality indices (C_i) and five functional indices (F_i) are defined. Degradations are identified respectively for each index, according to the degradation manual or to disfunctionality and importance is established for each one. According to the importance, a rank is given and the maximum of these ranks is deducted from 10, the highest value of each index. Progressively each value C_i and F_i is obtained.

Finally, the overall condition is expressed by the total condition index computed with formula $I_{ST} = \sum_{i=1}^{5} C_i + \sum_{i=1}^{5} F_i$. Based on this the technical class of the bridge is established and based on it one may decide the strategy of maintenance, reparation or rehabilitation of the bridge.

The advantage of the method is its simplicity. For calculation of the quality indices or the total condition index no complex formula is required.

The minus of the method is represented by the high level of subjectivism. For each item of degradation intervals of ranking are given without indicating precise criteria for each and every rank.

2.2. The method used by GETEC Company - France

Today, in France more evaluation methods are used for assessment of technical condition of highway bridges. This trend is due to the decentralization of the responsibility of administration of roads and bridges to the departmental level. Having the possibility to choose, administrators are in period of search of the best method. Further we will present the method developed by GETEC for departments Moselle and Haute Savoie from eastern France [3], which was used in our study.

In this method, the inspection of a bridge has more objectives: knowing the assets and completion of existing databases; assessment of structural condition and the cost required for its restoration; determining the priority and emergency for sorting the maintenance and rehabilitation interventions. Three categories of documents are drafted:

Notebook of the structure, a sort of identity card of the bridge which allows:

Identification;
Recording of technical and geometrical characteristics and functional importance;
Notes of the environment where the bridge is situated and the utilities suspended on the bridge;
Visualization of the structure using sketches and photos.

Visit notebook, a sort of bridge health card of the bridge that allows:

Recording and evaluation of the pathology affecting each sub-system;

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Giving a representative mark for condition (severity index);
Appreciation of complementary actions and intervention necessities;
Indication of emergency measures that must be done;
Evaluation of the cost of intervention works;
Visualization of the degradations using sketches and photos.
Synthesis notebook, which allow on the network:
Grouping the main evaluation data (IG, IF, etc.);
Combining the quality and functional parameters for determining the global mark for each structure (emergency index IU);
Classification of the structure according to this index for determining intervention priority;
Analysis of the assets.
As mentioned before, the method establishes for each element a severity index (IG) and a functional index (IF).
The severity index depends on degradation and importance of the element in the structure. Its computation has the following principle:

$$IG = \left[A \times \left(B + C + D + E\right) \times F\right]$$

IG takes values from 8 to 360, the higher the value the higher the degradation.

Table 1.Signification for the components of the severity index

	Component	Description
A	Importance level	It refers to the relative level of importance of the pathology in its position context. The level might be: low importance; medium importance; very important.
B	Exposure risk	It refers to the exposure of the pathology according to its location on the structure. It might be: low exposure; medium exposure; high exposure.
C	Environment effects	Which is the influence of the environment on the pathology. It might be: low importance; medium importance; very important.
D	Structure's condition	General condition of the structure surrounding the pathology. It might be: good condition; medium condition; bad condition.
E	Stress	Structure is in a stress condition that might induce fatigue phenomenon: less important; of medium importance; very important.
F	Group of risk	If affects elements that have little direct influence on limit state of the structure; If affects elements with important but indirect influence on limit state of the structure (on long term); If the pathology affects parts with direct influence on the limit state of the structure.

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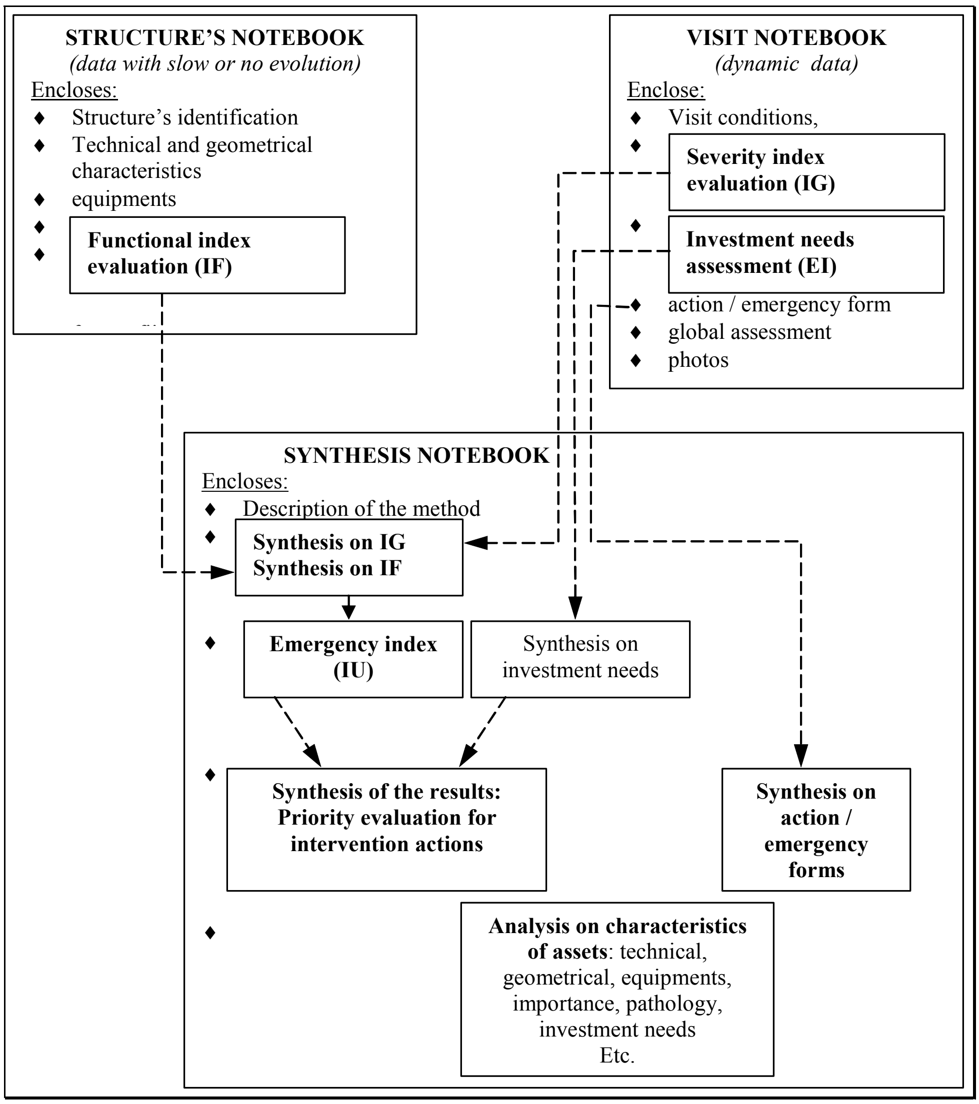

Fig. 1. The method for GETEC system

The functional index has the following computation principle $IF = N1 + N2 + N3 + N4 + N5$.

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Table 2. Significance of the components of the functional index

	Component	Meaning
N1	Function importance of the carried road	Inter-regional road; Main network; Other networks; Less used.
N2	Detour	Detour impossible; Detour longer than 10 km; Detour shorter than 10 km; Detour less than 1 km.
N3	Pedestrian traffic	Important pedestrian traffic; Few pedestrians; No pedestrians.
N4	Risks for the user in case of collapse	Important risks (habitation or traffic downstream); Medium risk (utilities posted on bridge) Low risk; No risk.
N5	Importance of the structure	According to its maximum spanning: The structure has a spanning has a spanning ≥50m The structure has a spanning has a spanning ≥10m The structure has a spanning has a spanning <10m but ≥2m The structure has a spanning has a spanning <2m and the hydrological area ≥2m^2

IF takes values from 0 to 20.

For each structure an emergency index IU might be computed: $IU = \sqrt{(18 \times IF)^2 + IG^2}$.

IU has the minimum value 8 and the maximum 509.

The parameter 18 has been chosen to keep IF in dimensional balance with IG. It is possible to use IU as a priority indicator.

3. CONCLUSIONS

The two methods presented hereby have common characteristic but also differences. They both respect the general criteria for condition assessment by visual inspection but they differ by complexity and detail level.

These two methods were used in parallel for over 1000 bridge in order to make a comparison between them for the future assessment of the pan/European corridors on the territory of Romania.

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Statistical aspects concerning the degradartion condition of bridges

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Sumarry

Starting from data acquired in inspections for assessment of technical condition, an analysis must always be conducted over the global situation encountered. Such operations must be considered in systematic procedures to obtain information that might lead us to calibration or, where necessary, to modification of the assessment system currently in use.

In theSETEOA program, a joint Romanian-French project that took place between 2001 and 2003, around 1200 bridges were included. Data and information acquired are further used to assessing the technical condition and works necessities for maintaining the bridges on the national roads network. They are also a start for developing the database of the bridge management system at National Administration of Roads (NAR). Under the statistical, the degradations observed were considered aspect through the two assessment systems used: AND522-2002 and GETEC coding.

Because the results processing is still in progress only 776 were included in the present analysis. As other information will be available the report will be updated. No significant differences are expected because we consider that the present sample include here is a representative sample for national level.

The distribution of bridge with respect to the main construction material is presented. As a confirmation of the sample being representative we may observe that the percentage of the bridges in reinforced concrete and pre-stressed concrete together represents 94.98%, which corresponds to the national percentage of 94.15% (52.82% reinforced concrete and 41.33 pre-stressed concrete) [1].

Predominance of these types of materials imposes some consequences: first the total length of the bridges is relatively low and second a pathology specific to these types of elements.

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INTRODUCTION

Starting from data acquired in inspections for assessment of technical condition, an analysis must always be conducted over the global situation encountered. Such operations must be considered in systematic procedures to obtain information that might lead us to calibration or, where necessary, to modification of the assessment system currently in use.

Further we will refer to the situation reached as a result of SETEOA program, a joint Romanian-French project that took place between 2001 and 2003. In this specialists from CESTRIN and GETEG were involved. Data and information acquired are further used to assessing the technical condition and works necessities for maintaining the bridges on the national roads network. They are also a start for developing the database of the bridge management system at National Administration of Roads (NAR).

GENERAL DATA

In this program around 1200 bridges were included. Because the results processing is still in progress only 776 were included in the present analysis. As other information will be available the report will be updated. No significant differences are expected because we consider that the present sample include here is a representative sample for national level.

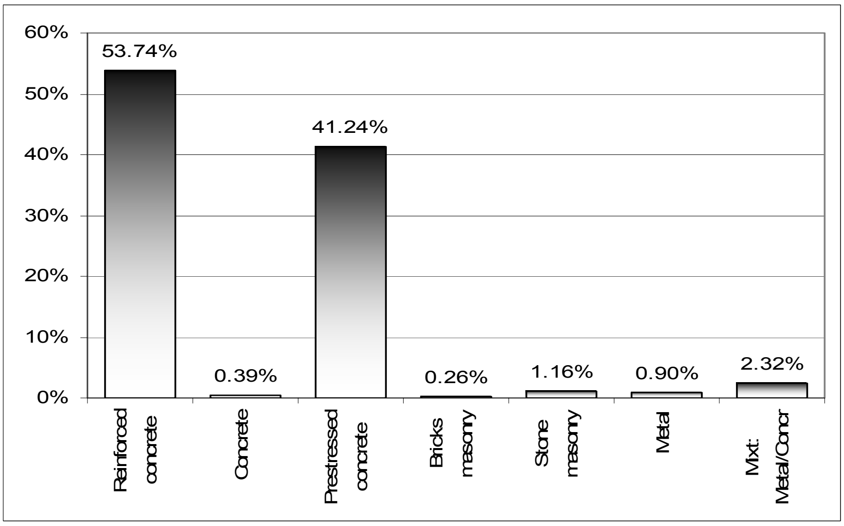

Fig. 1. Distribution of bridge on main material

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In figure 1 the distribution of bridge with respect to the main construction material is presented. As a confirmation of the sample being representative we may observe that the percentage of the bridges in reinforced concrete and pre-stressed concrete together represents 94.98%, which corresponds to the national percentage of 94.15% (52.82% reinforced concrete and 41.33 pre-stressed concrete) [1].

Predominance of these types of materials imposes some consequences: first the total length of the bridges is relatively low and second a pathology specific to these types of elements.

The figure 2 presents the distribution of bridge lengths as they were measured in this program.

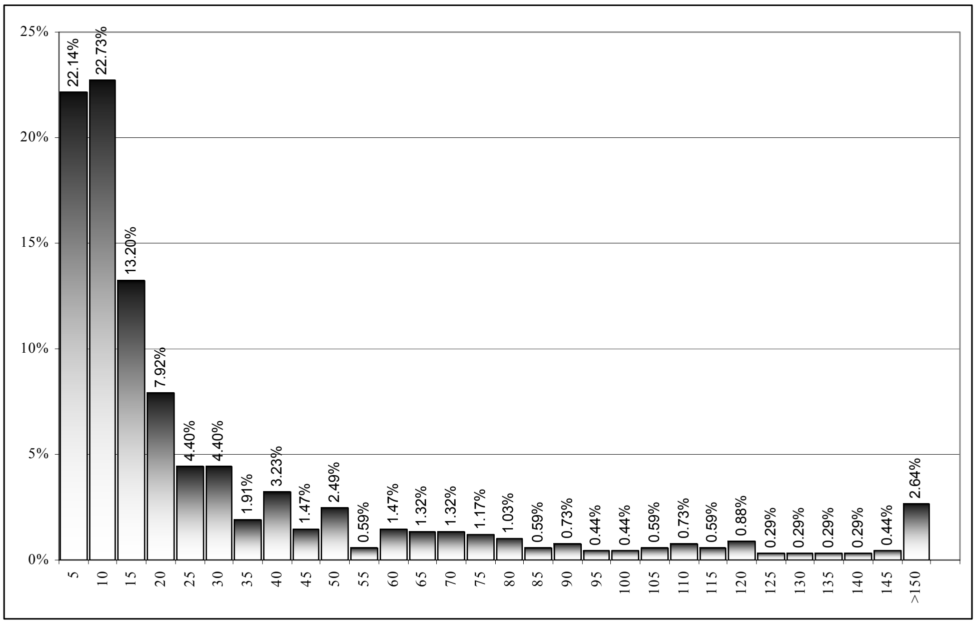

Fig. 2. Distribution of bridges with respect to the length

One might see that ~70% of the bridges have the length less than 30 meters.

DEGRADATION OCCURRENCE FREQUENCY

Under the statistical, the degradations observed were considered aspect through the two assessment systems used: AND522-2002 and GETEC coding.

For a better comparing we decided that the classification of degradations on constitutive elements to be performed according to Romanian method, this aspect being less important for the other method. The codes used are the ones described by the two methods. As presented in figures 3, 4, 5, and 6 GETEC system has a higher degree of details.

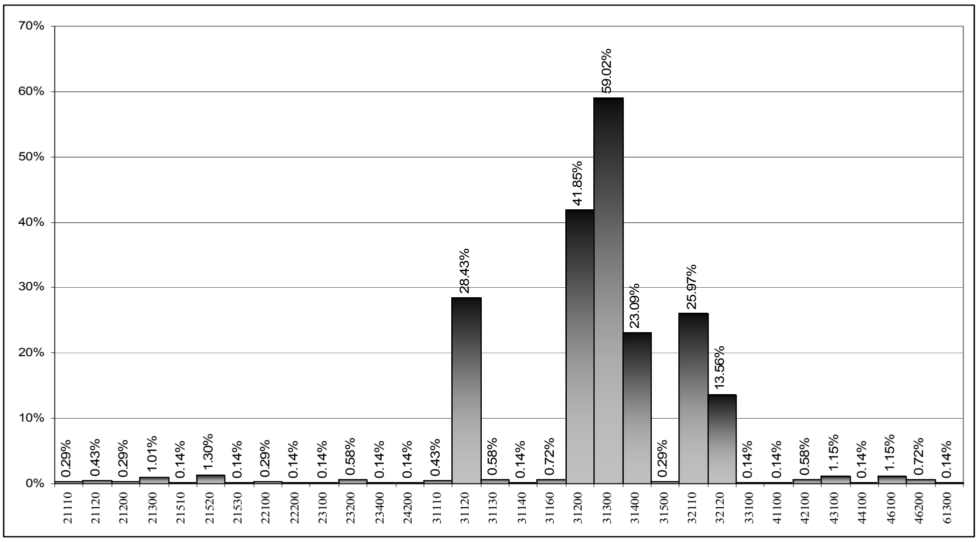

Fig. 3. The percentage of degradations on main elements of resistance according to French system

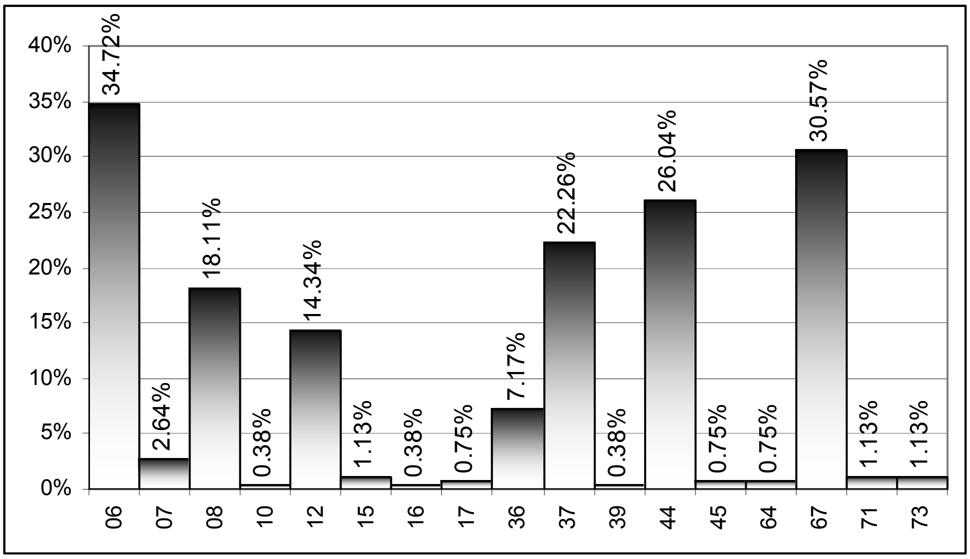

Fig. 4 The percentage of degradations on main elements of resistance according to Romanian system

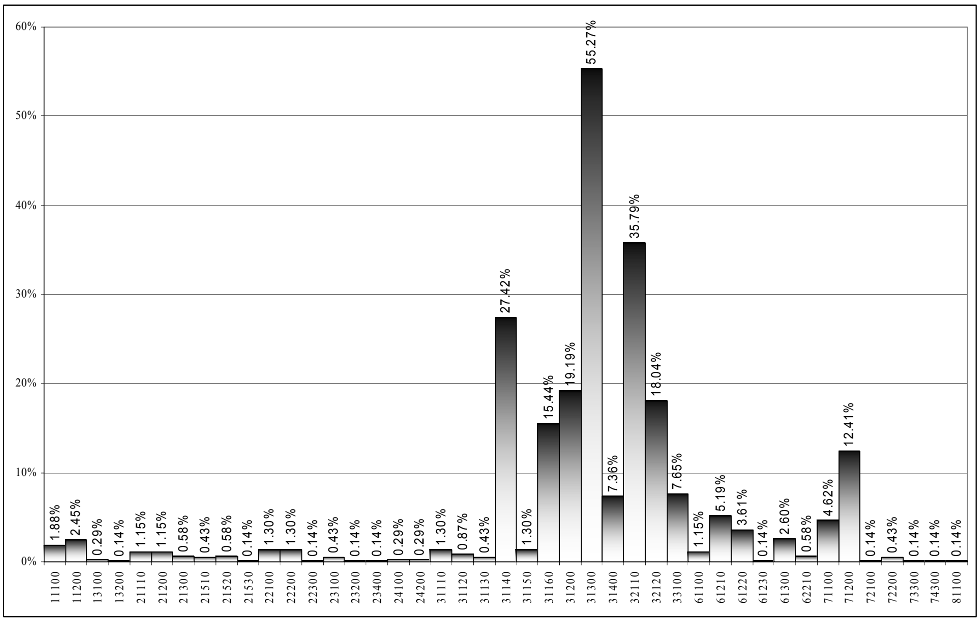

Fig. 5. The percentage of degradations on infrastructure according to French system

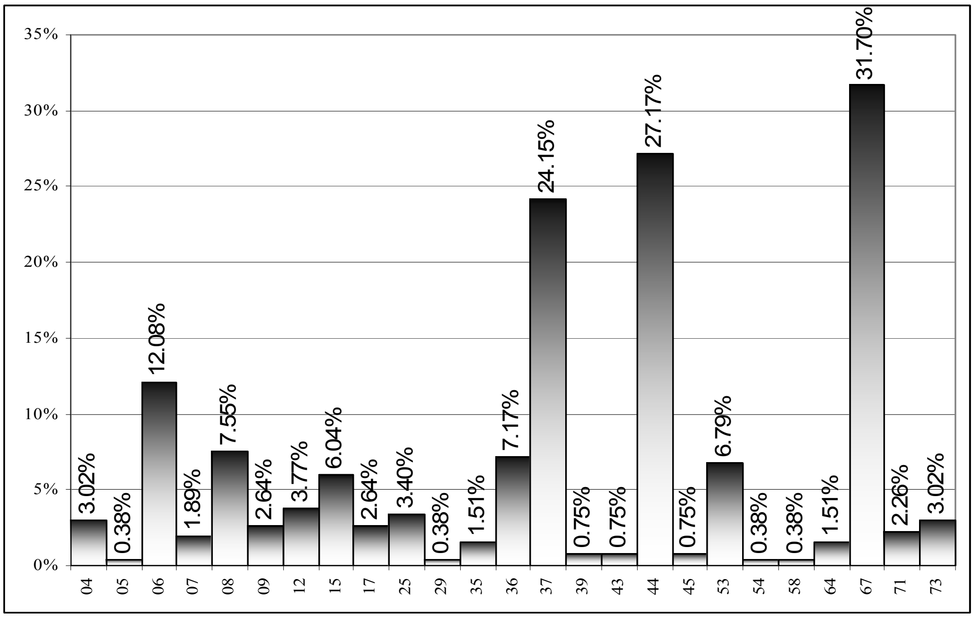

Fig. 6. The percentage of degradations on infrastructure according to Romanian system

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Of a special importance is the analysis of the degradation condition resulted after processing data about the quality of the material (for AND522 the sum of the degradation marks corresponding to *Ci*). This is further presented in fig. 7 and 8.

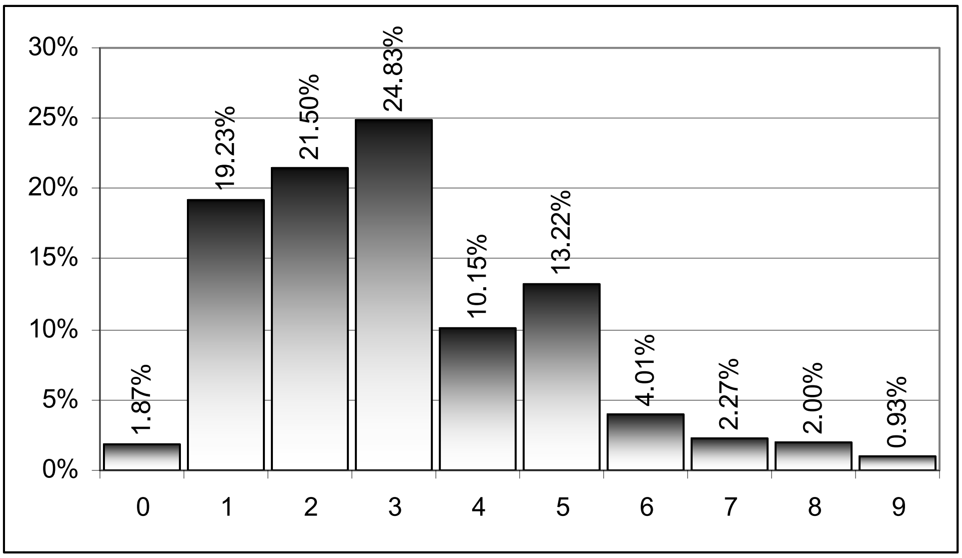

Fig. 7. Distributions of bridges with respect to the evaluation of the severity of the degradations according to the French system

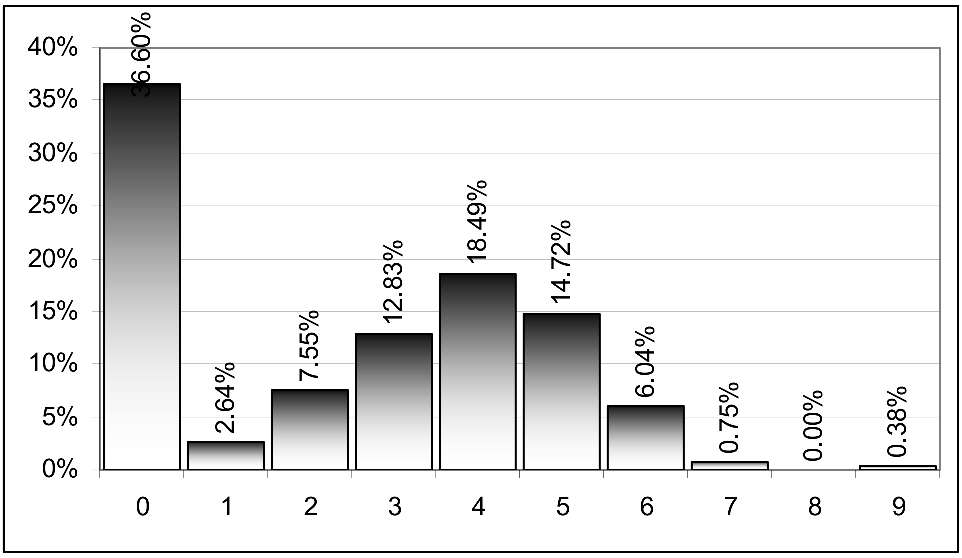

Fig. 8. Distributions of bridges with respect to the evaluation of the severity of the degradations according to the Romanian system

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We must note that the values from the horizontal axis from figures 7 and 8 represent conventional groups created by dividing each value scale in 10. Because the interval of values is different for the two methods we used different coefficients to bring them to the same scale. One may observe a significant difference in the allure of the distributions due to the differences in appreciation.

ENCOUNTERED DEGRADATIONS

A wide and diverse pathology was noticed. Following we present two examples.

Fig. 9a.

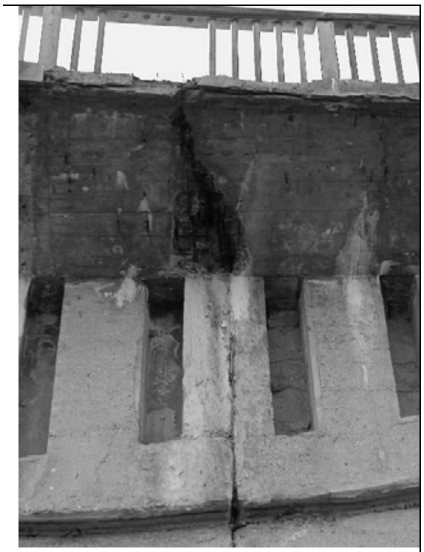
Fig. 9b.

Fig. 9c.

Fig. 10a.

Fig. 10b.

Fig. 10c.



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Optimal seismic response control of a long span cable-stayed bridge for a benchmark problem

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Summary

Structural Control has been growing in a fast pace as a subject contributing with means to alleviate the effects of harming loads. Correlated with other domains of human activity (electronics, automatic control, computer science, robotics, new materials, etc) important changes in philosophy and practice are recorded in Civil Engineering.

Because Structural Control is still an expensive approach to protect structures, in the views are structures as long cable-supported bridges and tall buildings that are vital for people and social activities, especially during and after strong winds or earthquakes.

This paper is showing a developing procedure of the classical method of optimal control. A first step into this development was previously done when the first author was showing that it is possible to lower the degree of arbitrariness for the coefficients in the weighting matrices based on energy considerations.

In this work the method is further developed when the attention is given to using reduced state models, which is a more realistic approach than using the full-state method as in previous works.

For the FEM model of the cable-stayed bridge given by an international benchmark, simulations have been performed using the method described above. External actions are three important strong earthquake acceleration records. The discrete time approach time and time-delay existing between the calculation and application of the control forces are taken into consideration. Also the process noise and measurements are considered. A simple predictive procedure is proposed.

Results of the application are showing that the time-response and also frequency responses are considerably reduced: stresses, bending moments, forces, displacements, velocities, and accelerations are kept into allowable limits. Based on the benchmark performance indexes the controlled system is very competitive.

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1. INTRODUCTION

Structural Control has been growing in a fast pace as a subject contributing with means to alleviate the effects of harming loads since its beginning [1]. Last years have shown a stress on researches about active control of large structures with many active devices. Optimal control has been used in [2] and [3] in a centralized setting. Decentralized controllers have been proposed in [4], [5] and [6]. Also, sliding mode control has been analyzed as a method to cope with uncertainties [7].

In the context of optimal control, an energy-based method for choosing the weighting parameters was developed [8]. The method is very convenient because it implies a simple way to set the control parameters. Using this method, it is possible to control large structures using many devices and, in this way, the efficiency and reliability of the control are highly increased. Then other authors adopted similar energy-based control strategies [9].

The international benchmark control problem for seismic response of cable-stayed bridges is used [10] to prove the efficiency of the proposed strategy, For this goal, the full state methodology for choosing the weighting control matrices is set up in order to adapt it to the needs of canonical modal transformation and model reduction procedures [11].

Therefore, realistic simulations with few measurement devices and reduced order estimator are performed. Simulations take into account the discrete-time aspects of a real application, along with process noise, measurement noise, and application of control forces time delay. In this paper, the method is further improved and tested through the use of a simple predictive method for the measurements, in order to avoid time delays. Comparisons of the predictive strategy and non-predictive control with the benchmark sample control strategy are done. Results show good behavior of the proposed control methodologies according to a set of evaluation criteria established by the benchmark.

2. METHODOLOGY. OPTIMAL CONTROL APPROACH FOR REDUCED STATE SYSTEMS

Optimal active control is a time domain strategy that is appropriate for controlling the response of structures subjected to strong earthquakes, [12]. The strategy allows minimizing the induced structural energy [13].

In Structural Control, the state equation of motion for a n degree of freedom controlled system under seismic action is:

$$\dot{\mathbf{x}}(t) = \mathbf{A}\mathbf{x}(t) + \mathbf{B}\mathbf{u}(t) + \mathbf{h}\ddot{x}_g(t), \quad \mathbf{x}(0) = \mathbf{x}_0 \tag{1}$$

where **x***(t)* is the 2*n*-dimensional state, and $\mathbf{A}$, $\mathbf{B}$ are appropriate matrices.

Setting the control actions as $\mathbf{u}(t) = -\mathbf{K}\mathbf{x}(t)$, the goal of the method is to obtain the feedback gain matrix **K** to minimize a performance index *J* defined by

$$J = \frac{1}{2}\int_0^\infty \left[\mathbf{x}'(t)\mathbf{Q}\mathbf{x}(t) + \mathbf{u}'(t)\mathbf{R}\mathbf{u}(t)\right]dt \tag{2}$$

where **Q** and **R** are weighting matrices, 2*n*×2*n*-dimensional and *m*×*m*-dimensional, respectively; and *m* is the number of the actuators. Minimization of the performance criterion (2) implies to solve the Riccati equation

$$\mathbf{PA} - \mathbf{PBR}^{-1}\mathbf{B}'\mathbf{P} + \mathbf{A}'\mathbf{P} + \mathbf{Q} = \mathbf{0} \tag{3}$$

Then, the control gain matrix is $\mathbf{K} = \mathbf{R}^{-1}\mathbf{B}'\mathbf{P}$.

Appropriate settings can be found for the full states-based matrices **Q** and **R**. For example, if $\mathbf{Q} = \mathrm{diag}(\mathbf{K}_1, \mathbf{M}_1)$, then the first term in the brackets of Equation (2) is an energy expression and therefore Equation (2) leads to minimization of the energy of the structural response. Matrix **R** can be set as $\mathbf{R} = r\mathbf{I}$, where **I** is the identity matrix and *r* is a scalar, the unique parameter to be determined [13].

Usually, only few measurements are available. In this case, the output of the system, **y**, is expressed through a second equation complementing the state-space model, in the form:

$$\begin{cases} \dot{\mathbf{x}} = \mathbf{A}\mathbf{x} + \mathbf{B}\mathbf{u} + \mathbf{h}\ddot{x}_g \\ \mathbf{y} = \mathbf{C}\mathbf{x} + \mathbf{D}\mathbf{u} \end{cases} \tag{4}$$

where **C** is a *p*×2*n* measurement matrix and **D** is the *p*×*m* matrix showing the influence of the control forces on the output.

A first step to avoid the inconvenience of using all states is to apply a canonical transformation $\mathbf{x}_c = \mathbf{T}_c\mathbf{x}$ or $\mathbf{x} = \mathbf{T}_c^{-1}\mathbf{x}_c = \mathbf{P}_c\mathbf{x}_c$ based on the eigenvector matrix $\mathbf{P} = \mathbf{T}_c^{-1}$. This way, the system (4) takes the new form:

$$\begin{cases} \dot{\mathbf{x}}_c = \mathbf{A}_c\mathbf{x}_c + \mathbf{B}_c\mathbf{u} + \mathbf{h}_c\ddot{x}_g \\ \mathbf{y} = \mathbf{C}_c\mathbf{x}_c + \mathbf{D}\mathbf{u} \end{cases} \tag{5}$$

where $\mathbf{A}_c = \mathbf{P}^{-1}\mathbf{A}\mathbf{P}$, $\mathbf{B}_c = \mathbf{P}_c^{-1}\mathbf{B}$, $\mathbf{C}_c = \mathbf{P}_c^{-1}\mathbf{C}$, and $\mathbf{h}_c = \mathbf{P}_c^{-1}\mathbf{h}$.

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The second step, based on controllability and observability gramians [14,15], is to use a state coordinate transformation matrix $\overline{\mathbf{T}}$ applied to the system (5), i.e., $\overline{\mathbf{x}} = \overline{\mathbf{T}}\mathbf{x}_c$. The resulting system is a balanced system:

$$\begin{cases} \dot{\overline{\mathbf{x}}} = \overline{\mathbf{A}}\overline{\mathbf{x}} + \overline{\mathbf{B}}\mathbf{u} + \overline{\mathbf{h}}\ddot{x}_g \\ \mathbf{y} = \overline{\mathbf{C}}\overline{\mathbf{x}} + \mathbf{D}\mathbf{u} \end{cases} \tag{6}$$

where $\overline{\mathbf{A}} = \overline{\mathbf{T}}\mathbf{A}_c\overline{\mathbf{T}}^{-1}$, $\overline{\mathbf{B}} = \overline{\mathbf{T}}\mathbf{B}_c$, $\overline{\mathbf{C}} = \mathbf{C}_c\overline{\mathbf{T}}^{-1}$, and $\overline{\mathbf{h}} = \overline{\mathbf{T}}\mathbf{h}_c$. If only the first most significant q states are retained for the structural response, the system (6) can be rewritten in the form:

$$\begin{cases} \begin{Bmatrix} \dot{\overline{\mathbf{x}}}_1 \\ \mathbf{0} \end{Bmatrix} = \begin{bmatrix} \overline{\mathbf{A}}_{11} & \overline{\mathbf{A}}_{12} \\ \overline{\mathbf{A}}_{21} & \overline{\mathbf{A}}_{22} \end{bmatrix} \begin{Bmatrix} \overline{\mathbf{x}}_1 \\ \overline{\mathbf{x}}_2 \end{Bmatrix} + \begin{Bmatrix} \overline{\mathbf{B}}_1 \\ \overline{\mathbf{B}}_2 \end{Bmatrix} \mathbf{u} + \begin{Bmatrix} \overline{\mathbf{h}}_1 \\ \overline{\mathbf{h}}_2 \end{Bmatrix} \ddot{x}_g \\ \mathbf{y} = \begin{bmatrix} \overline{\mathbf{C}}_1 & \overline{\mathbf{C}}_2 \end{bmatrix} \begin{Bmatrix} \overline{\mathbf{x}}_1 \\ \overline{\mathbf{x}}_2 \end{Bmatrix} + \mathbf{D}\mathbf{u} \end{cases} \tag{7}$$

where $\overline{\mathbf{x}}_1$ are the states to be retained, a q-dimensional vector. $\overline{\mathbf{x}}_2$ are the states to be eliminated, $\overline{\mathbf{x}}_2 = -\overline{\mathbf{A}}_{22}^{-1}\overline{\mathbf{A}}_{21}\overline{\mathbf{x}}_1 - \overline{\mathbf{A}}_{22}^{-1}\overline{\mathbf{B}}_2\mathbf{u} - \overline{\mathbf{A}}_{22}^{-1}\overline{\mathbf{h}}_2\ddot{x}_g$. This way, the reduced state system is:

$$\begin{cases} \dot{\mathbf{x}}_r = \mathbf{A}_r\mathbf{x}_r + \mathbf{B}_r\mathbf{u} + \mathbf{h}_r\ddot{x}_g \\ \mathbf{y} = \mathbf{C}_r\mathbf{x}_r + \mathbf{D}_r\mathbf{u} + \mathbf{h}_y\ddot{x}_g \end{cases} \tag{8}$$

where:, $\mathbf{A}_r = \overline{\mathbf{A}}_{11} - \overline{\mathbf{A}}_{12}\overline{\mathbf{A}}_{22}^{-1}\overline{\mathbf{A}}_{21}$, $\mathbf{B}_r = \overline{\mathbf{B}}_1 - \overline{\mathbf{A}}_{12}\overline{\mathbf{A}}_{22}^{-1}\overline{\mathbf{B}}_2$, $\mathbf{h}_r = \overline{\mathbf{h}}_1 - \overline{\mathbf{A}}_{12}\overline{\mathbf{A}}_{22}^{-1}\overline{\mathbf{h}}_2$, $\mathbf{C}_r = \mathbf{C}_1 - \mathbf{C}_2\overline{\mathbf{A}}_{22}^{-1}\overline{\mathbf{A}}_{21}$, $\mathbf{D}_r = \mathbf{D} - \mathbf{C}_2\overline{\mathbf{A}}_{22}^{-1}\overline{\mathbf{B}}_2$, $\mathbf{h}_y = -\mathbf{C}_2\overline{\mathbf{A}}_{22}^{-1}\overline{\mathbf{h}}_2$, and $\mathbf{x}_r = \overline{\mathbf{x}}_1$.

Therefore, for the system (8), the index to be minimized is:

$$J = \frac{1}{2}\int_0^{t_f} \left[\mathbf{x}_r'\mathbf{Q}_e\mathbf{x}_r + \mathbf{u}'(\mathbf{R} + \mathbf{R}_e)\mathbf{u} + 2\mathbf{x}_r'\mathbf{N}_e\mathbf{u}\right] dt \tag{9}$$

where $\mathbf{A}_e = \left[\mathbf{I} \quad -\overline{\mathbf{A}}_{22}^{-1}\overline{\mathbf{A}}_{21}\right]'$, $\mathbf{B}_e = \left[\mathbf{0} \quad -\overline{\mathbf{A}}_{22}^{-1}\overline{\mathbf{B}}_2\right]'$, $\mathbf{N}_e = \mathbf{A}_e'\left(\overline{\mathbf{T}}^{-1}\right)'\mathbf{P}_c'\mathbf{Q}\mathbf{P}_c\overline{\mathbf{T}}^{-1}\mathbf{B}_e$ $\mathbf{Q}_e = \mathbf{A}_e'\left(\overline{\mathbf{T}}^{-1}\right)'\mathbf{P}_c'\mathbf{Q}\mathbf{P}_c\overline{\mathbf{T}}^{-1}\mathbf{A}_e$, and $\mathbf{R}_e = \mathbf{B}_e'\left(\overline{\mathbf{T}}^{-1}\right)'\mathbf{P}_c'\mathbf{Q}\mathbf{P}_c\overline{\mathbf{T}}^{-1}\mathbf{B}_e$.

Note that, because of the above transformations, Equation (9) is an approximation of the Equation (2). The corresponding Riccati equation is then:

$$\mathbf{PA} - (\mathbf{PB} + \mathbf{N}_e)(\mathbf{R} + \mathbf{R}_e)^{-1}(\mathbf{B}'\mathbf{P} + \mathbf{N}_e') + \mathbf{A}'\mathbf{P} + \mathbf{Q}_e = \mathbf{0} \tag{10}$$

and the gain matrix is expressed by: $\mathbf{K} = (\mathbf{R} + \mathbf{R}_e)^{-1}(\mathbf{B}'\mathbf{P} + \mathbf{N}_e')$.

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For real applications, the model from Equation (8) can be adapted into the form:

$$\begin{cases} \dot{\mathbf{x}}_r = \mathbf{A}_r\mathbf{x}_r + \mathbf{B}_r\mathbf{u} + \mathbf{h}_r\ddot{x}_g + \mathbf{G}\mathbf{w} \\ \mathbf{y}_m = \mathbf{C}_r\mathbf{x}_r + \mathbf{D}_r\mathbf{u} + \mathbf{h}_y\ddot{x}_g + \mathbf{H}\mathbf{w} + \mathbf{v} \end{cases} \tag{11}$$

where **w** and **v** are the process noise and the measurement noise vectors respectively; **G** and **H** are distribution matrices and $\mathbf{y}_m$ is the measured output vector.

The states of the system (11) can be estimated from the measured outputs using for example a Kalman filter [16]:

$$\dot{\hat{\mathbf{x}}}_r = \mathbf{A}\hat{\mathbf{x}}_r + \mathbf{B}\mathbf{u} + \mathbf{L}\left(\mathbf{y}_m - \mathbf{C}_r\hat{\mathbf{x}}_r - \mathbf{D}_r\mathbf{u}\right) \tag{12}$$

where $\hat{\mathbf{x}}_r$ is the vector of estimated states and **L** is the filter gain matrix deduced from solving also a Riccati equation. Then the control forces are: $\mathbf{u} = -\mathbf{K}\hat{\mathbf{x}}_r$.

All the procedure shown above is formulated in continuous time. The application in the next section is using the discrete time version of the method as the practice requires. A time delay between the computation of control forces and their application is also considered. Supposing a time delay equal to the sampling time, Δt, the control forces are applied at the time, t_{i+1}. This is one step after the real measuring time, t_i, when the measurement vector, $\mathbf{y}_{m,i}$, was obtained.

Also, a very simple linear predictive scheme can be applied: the current measurement vector, $\mathbf{y}_{m,i}$, is considered an average from the previous one, $\mathbf{y}_{m,i-1}$, and the future one, $\mathbf{y}_{m,i+1}$, i.e.

$$\mathbf{y}_{m,i+1} = 2\mathbf{y}_{m,i} - \mathbf{y}_{m,i-1} \tag{13}$$

This way, the estimated states from Equation (12) are deduced based on the predicted measurement vector defined by Equation (13).

3. APPLICATION TO THE BENCHMARK MODEL

The control procedure described previously is applied to the model of a cable-stayed bridge, Cape Girardeau over Mississippi River, that is the object of a benchmark problem [10]. The main span of the bridge is 350.6 m with the side spans of 142.7 m in length. It has a total of 128 cables that connect the 29.3 m wide deck with the towers, 100 m and 105 m tall. Figure 1 shows the bridge FEM model (left) and the three applied actions, El-Centro NS, 1940, Mexico City, 1985, and Gebze NS, 1999, (right).

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Statical reduction of the initial FEM model leads to a 419 dynamical degrees of freedom system. The system is reduced to a system with 30 states, as it is done in the benchmark sample scheme, for reasons of comparisons.

For the digital implementation, continuous to discrete-time signal converters are included. Four longitudinal direction accelerometers are placed on the on tops of the towers and one is located in the mid span. Four sensors measuring displacements were located between the towers and the deck [10].

In order to evaluate and compare the results of the proposed control strategies, the benchmark establishes 18 performance criteria. First six criteria refer to peak responses; the criteria from seven to eleven are related to normed responses, while the criteria twelve to eighteen are concerned to control strategy.

In order to choose a suitable value for the weighting matrix **R**, based on an unique scalar *r*, comparisons between the responses or criteria heve been performed. For this application, in the case of non-predictive control option, the scalar *r* took 14 different values within the interval [1.0e19, 1.0e22]. In the case of predictive strategy, the scalar *r* took 16 values in the same interval.

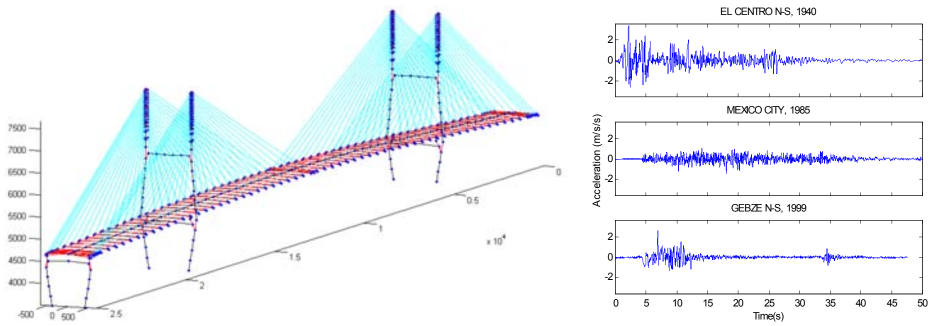

Fig.1. FEM model of the bridge (left) and the three external actions (right)

As an example of the design strategy, in Figure 2 the variation of the criteria number three (relative, maximum overturning moment for towers) as a function of the scalar *r* are presented. The values of these criteria given in the benchmark sample solution are also presented (as horizontal lines of the same type) for comparison. It can be seen a non-constant behavior of the results (values of criteria). In addition, this behavior is still a function of the three external actions (earthquakes), so different in content and effects.

Following the previous paragraphs' ideas, Table 1 shows a numerical comparison for the different results obtained for the first 15 criteria under the selected values for *r*. For each of the three earthquake actions there are shown three different columns with criteria values. The case *a)* refers to the benchmark given sample control (as the base of comparison). The case *b)* refers to applying the strategy

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proposed in the section above without prediction, with r = 3.0e20, while *c)* refers to the case using prediction with r = 7.5e20.

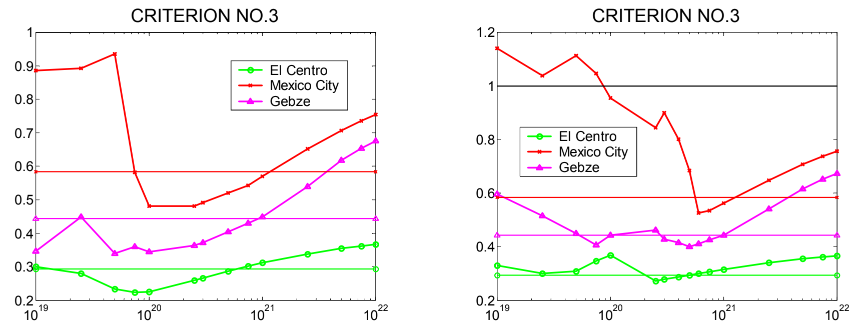

Fig.2. Criterion no.3 for the cases without/with prediction (left/right)

From Table 1, important observations can be withdrawn.

Table 1. Comparison of the benchmark (a) and the obtained results (b,c)

Crit.	No-ctrl.	El Centro Earthquake			Mexico Earthquake			Gebze Earthquake		
		a)	b)	c)	a)	b)	c)	a)	b)	c)
J_1	1.0	0.3868	0.3517*	0.3764**	0.4582	0.4366**	0.4201*	0.4540	0.4061*	0.4102**
J_2	1.0	1.0681	1.0213*	1.0534**	1.3693	1.2048*	1.3030**	1.3784	1.2262*	1.2532**
J_3	1.0	0.2944**	0.2673*	0.3066	0.5836	0.4918*	0.5345**	0.4434	0.3725*	0.4260**
J_4	1.0	0.6252	0.5883*	0.5625**	0.6140**	0.5786*	0.6422	1.2246*	1.2673**	1.6362
J_5	.8029 .1481 .3832	0.1861*	0.1944**	0.2045	0.0775	0.0694*	0.0740**	0.1481*	0.1629**	0.1991
J_6	1.0000	1.2006**	1.1730*	1.3164	2.3317*	2.3472**	3.1808	3.5640*	3.8828**	5.3403
J_7	1.0000	0.2257	0.2118*	0.2177**	0.3983	0.3649*	0.3695**	0.3231	0.2986*	0.3009**
J_8	1.0000	1.1778	1.0617*	1.1333**	1.2118	1.0795*	1.1615**	1.4371**	1.4026*	1.6618
J_9	1.0000	0.2665	0.2401*	0.2599**	0.4192**	0.3875*	0.4209	0.4552**	0.4520*	0.5246
J_{10}	1.0000	0.8813**	0.8056*	0.8946	1.1067**	1.0863*	1.2066	1.4570*	1.7621**	2.2947
J_{11}	0.0867 0.0225 0.0423	0.0280	0.0245*	0.0272**	0.0103	0.0092*	0.0100**	0.0171**	0.0167*	0.0194
J_{12}	0.0000	0.0016*	0.0019**	0.0016*	0.0006*	0.0006*	0.0005**	0.0017**	0.0018	0.0015*
J_{13}	0.0000	0.7883**	0.7702*	0.8643	1.1742*	1.1820**	1.6018	1.9541*	2.1288**	2.9279
J_{14}	0.0000	0.0027*	0.0027*	0.0020**	0.0018	0.0016*	0.0011**	0.0074	0.0071**	0.0058**
J_{15}	0.0000	0.0004**	0.0004**	0.0003*	0.0002**	0.0002**	0.0001*	0.0007**	0.0007**	0.0006*
Total 1st pos.		3 (20%)	12 (80%)	2 (13%)	3 (20%)	11 (73%)	2 (13%)	5 (34%)	7 (47%)	2 (13%)
Total 2nd pos.		5 (33%)	3 (20%)	8 (54%)	4 (27%)	4 (27%)	8 (54%)	5 (33%)	7 (47%)	5 (33%)
Total 3rd pos.		7 (47%)	0 (0%)	5 (33%)	8 (53%)	0 (0%)	5 (33%)	5 (33%)	1 (6%)	8 (54%)

* Best value (first position); ** Second position.

Therefore the superiority of the non-predictive strategy is strongly shown, specially by As a general idea, the strategy *b)* (non-predictive) looks the most successful while the predictive strategy *c)* is better than the benchmark example, *a)*. The explanation for this is that the prediction was acceptable for the displacement sensors and not acceptable for the acceleration sensors due to the high dynamics of these sensors' signals and the relatively long sampling time (0.02 sec.). The first 6 criteria, i.e. reduction of the maximum responses for base shear, shear at deck level, overturning moment, moment at the deck level, cable tension, displacement at the abutment level.

4. CONCLUSIONS

In this paper a previous work, [11], is further improved to the need of more realistic applications. The weighting matrix **Q** choice is done on energy-based procedure for the full state system. Since the measurements and the estimators cannot assure the knowledge or approximation of too many states, a reduced order model is employed [14,15]. The matrix **Q** is reduced following similar transformations, as the system itself. Simple prediction scheme for measurements is proposed, in order to avoid delays in applying control forces. A finite element model of a bridge proposed as a benchmark problem for structural control under seismic actions [10] is used for application. Simulations take into account the discrete-time aspects of a real application, along with process noise and measurement noise. Good behavior of the controlled system according to the benchmark evaluation criteria set, especially for the case without prediction, is noted and discussed.

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Analysis of dynamic behaviour under traffic loads of a strengthened old steel bridge

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Summary

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-the century begin: the main truss girder with parabolic upper chord and downward cross stud, longitudinal and transverse floor beams (stringers and cross girders) at lower chord, a ZORÉS profiled deck covered by road's system and lower and partially upper wind bracings. In figure 1 is presented the general view of bridge and in figure 2 some pictures of old structure.

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.

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.

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.

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Analysis of dynamic behavior of a steel bridge

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$^{\text{-the}}$ century begin: the main truss girder with parabolic upper chord and downward cross stud, longitudinal and transverse floor beams (stringers and cross girders) at lower chord, a ZORÉS profiled deck covered by road's system and lower and partially upper wind bracings. In figure 1 is presented the general view of bridge and in figure 2 some pictures of old structure.

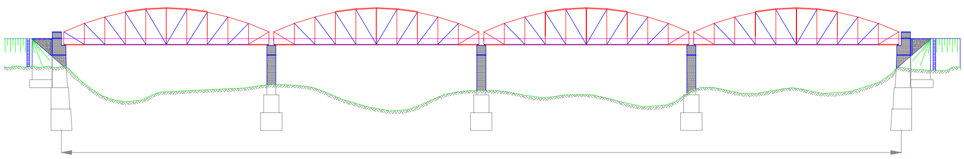

Figure 1. General view of Savarsin bridge

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].

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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 riveted steel element, a new weld able flange was attached to stringers and crossbeams using high strength bolts. In the same time the cross girder 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.

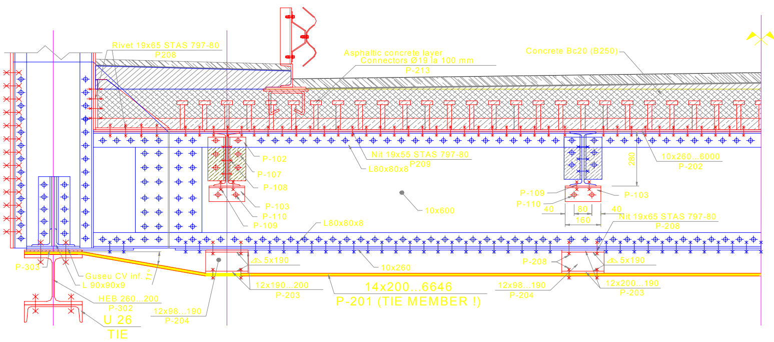

Figure 3. Floor beams cross-section

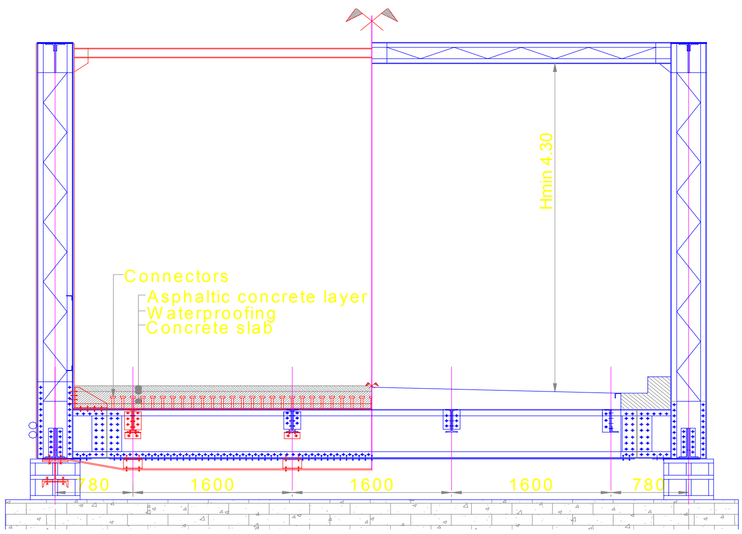

Figure 4. Bridge cross section

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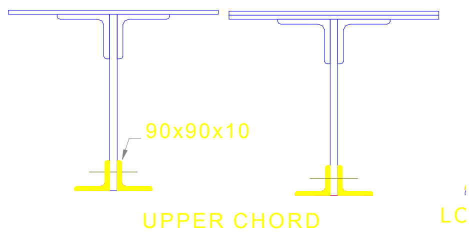
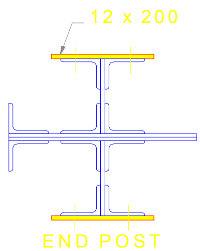
Main girders rehabilitation. For main girders a mixed reinforcing method was applied. That means: a) the direct strengthening of single wall cross-section of upper chord by adding two angle iron so it increase the member's second moment of area and reduce the element slenderness, specially from minor 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 wind bracing which has been strongly crooked and damaged by over roadway clearance carriages (e.g. trucks loaded with big saw logs).



a)



b)

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, wind bracing, 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

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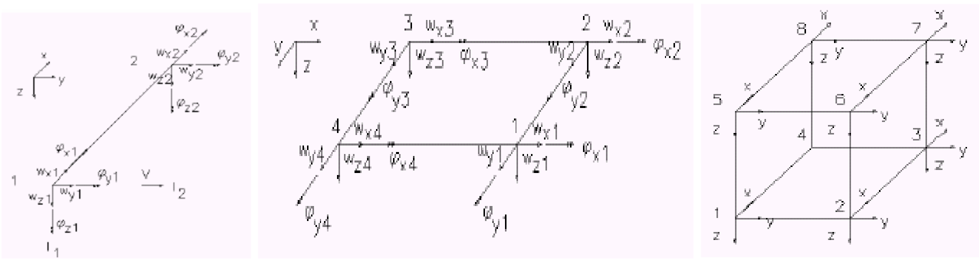
normal, shearing and bending general load transfer. This element can be modelled also as triangle. The square PLATE element is a very complex element, there it 20 knot degrees of freedom and thus a 20x20-Elementmatrix possesses.

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.

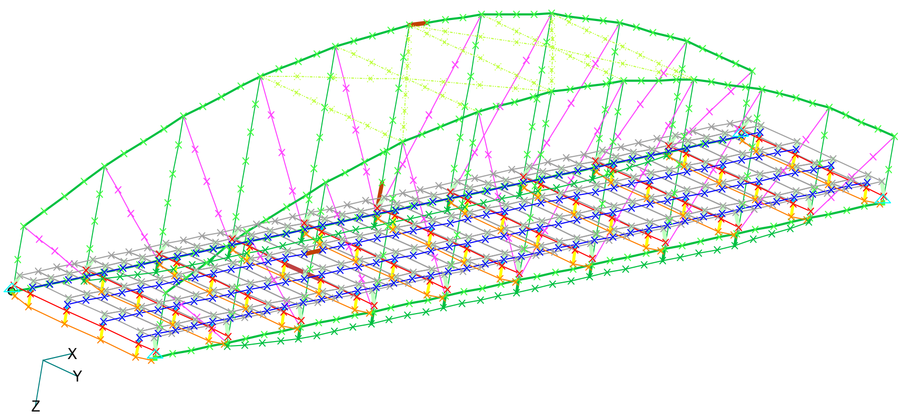


Figure 7. The entire model of reinforced bridge structure

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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).

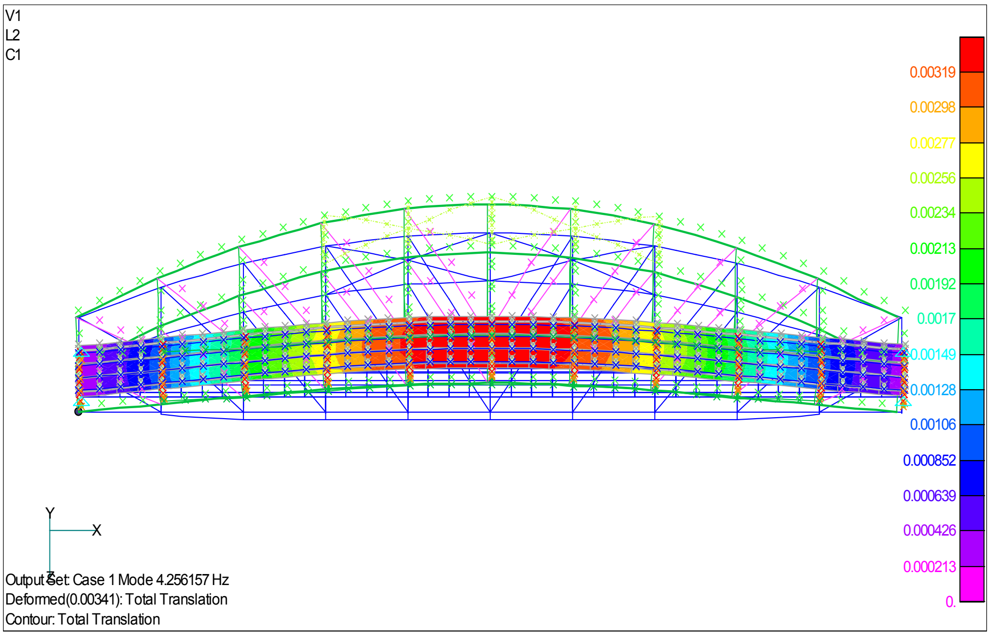

Figure 8. The first eigenmode and corresponding frequency

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 well known 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.

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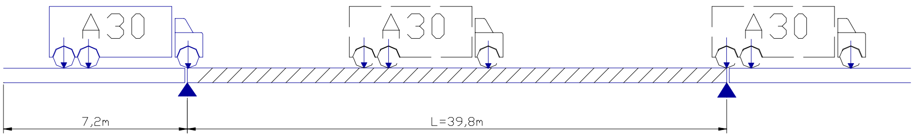

Figure. 9. Run travel of the A30-truck over the bridge

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.

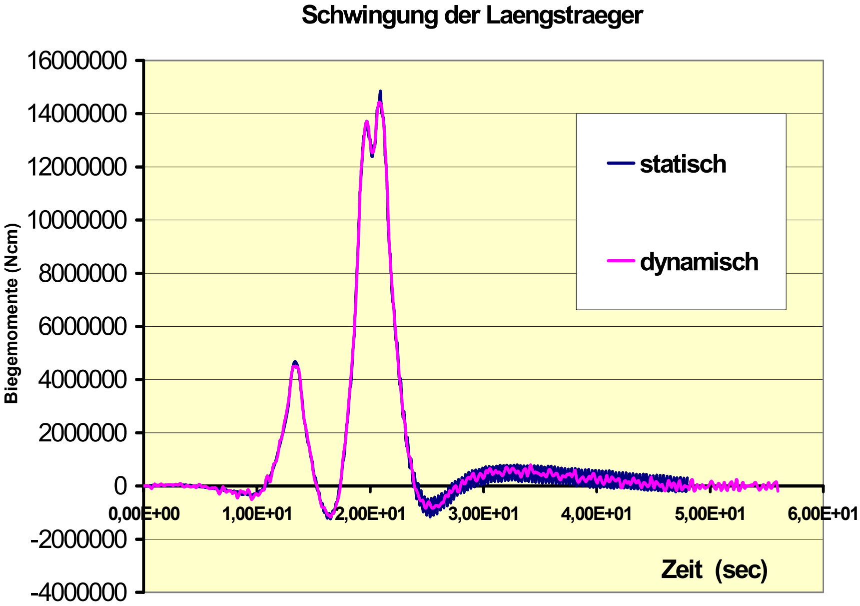

Figure 10. The dynamic effect of stringer

Additional analyses are recommended for the future with considering also:

roadway imperfections those experimentally measured knew;

truck natural oscillations.

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The influence of different types of hangers system of steel arch bridges

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Summary

The aim of this paper is to analyse the lateral stability of the Medgidia arch steel bridge upporting live loads consisting of Class E of vehicles (A30, V80) and having different types of hangers systems. For the analysis, the arch bridge was considered in the existing form, in which he was erected and having a Langer system for hangers and supplementary also two Nielsen systems are taked into account: the first system consist in inclined hangers forming a triangular net and the second one consist in inclined crossing hangers. The Lusas Finite Element System was used to perform all the analysis presented in this paper.

In order to establish which is the influence of each hangers system on the general stability of the arch bridge, many types of analysis are made. In the first stage a linear elastic analysis is performed. Further, other types of analysis are made: a linear buckling analysis which give information about the maximum load that can be supported prior to structural instability and a geometrically nonlinear analysis which take into account the effect of structural deformation on structural stiffness. The defomed mesh of the structure resulting from the linear elastic analysis was considered as the starting point for the geometrically nonlinear analysis.

The vehicles acting on the bridge are placed on the deck taking into account the romanian norm specifications from the for road bridges, 3221-86. For this reason, the load consisting în many rows of A30 vehicles has a non-symmetrical position and the load corresponding to V80 vehicle has a symmetrical position on the bridge deck.

Performing a step by step geometrically nonlinear analysis, a series of P-Δ (load-displacement) curves are ploted and from these, the critical buckling load of the arches corresponding to each type of hangers system could be computed.

Finally a comparaison between these load-displacement curves is made, which could give some informations about the influence of each type of hangers system on the general stability of this arch steel bridge.

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I. R. Răcănel

THE INFLUENCE OF DIFFERENT TYPES OF HANGERS SYSTEM OF STEEL ARCH BRIDGES

The aim of this paper is to analyse the lateral stability of the Medgidia arch steel bridge upporting live loads consisting of Class E of vehicles (A30, V80) and having different types of hangers systems. For the analysis, the arch bridge was considered in the existing form, in which he was erected and having a Langer system for hangers and supplementary also two Nielsen systems are taked into account: the first system consist in inclined hangers forming a triangular net and the second one consist in inclined crossing hangers. The Lusas Finite Element System was used to perform all the analysis presented in this paper.

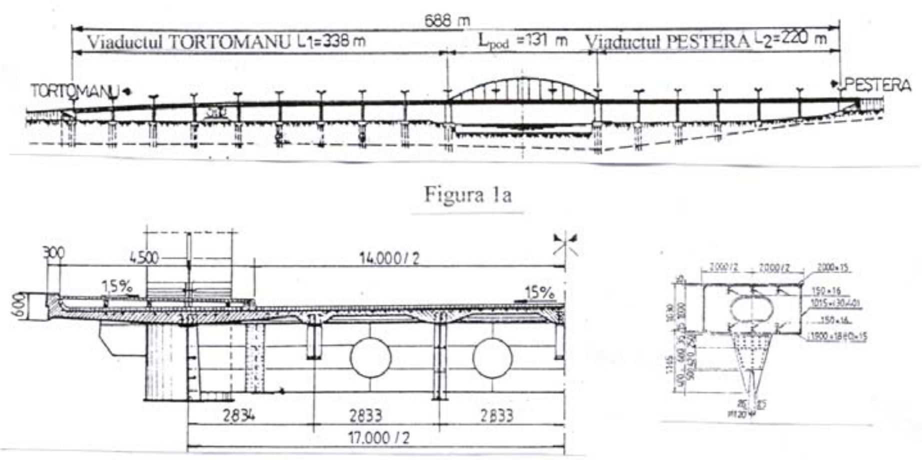

Fig. 1.

In order to establish which is the influence of each hangers system on the general stability of the arch bridge, many types of analysis are made. In the first stage a linear elastic analysis is performed. Further, other types of analysis are made: a linear buckling analysis which give information about the maximum load that can be supported prior to structural instability and a geometrically nonlinear analysis which take into account the effect of structural deformation on structural stiffness. The defomed mesh of the structure resulting from the linear elastic analysis was considered as the starting point for the geometrically nonlinear analysis.

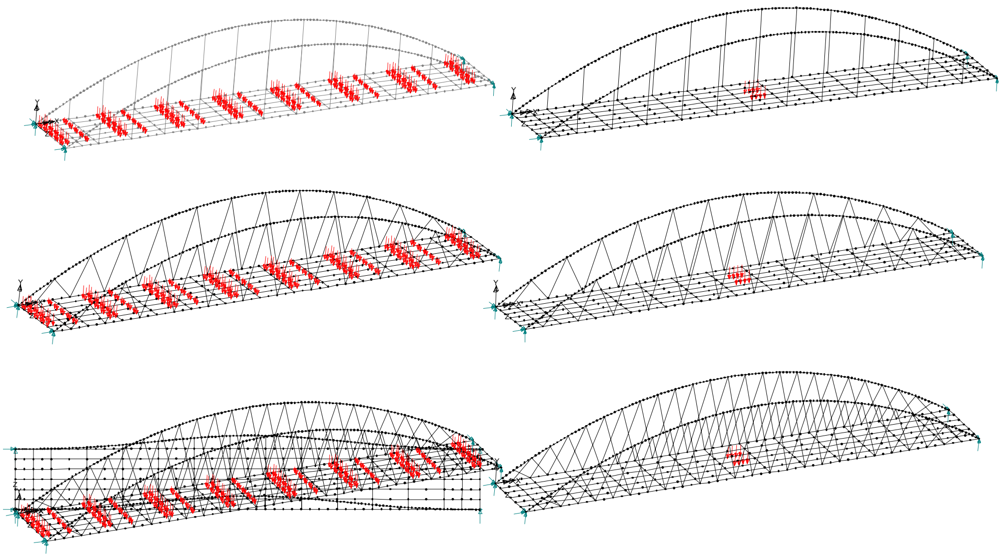

Fig. 2.

The vehicles acting on the bridge are placed on the deck taking into account the romanian norm specifications from the for road bridges, 3221-86. For this reason, the load consisting în many rows of A30 vehicles has a non-symmetrical position and the load corresponding to V80 vehicle has a symmetrical position on the bridge deck.

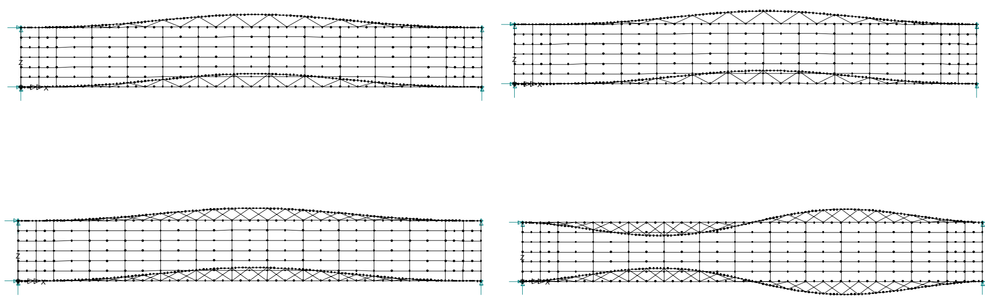

Fig. 3.

Performing a step by step geometrically nonlinear analysis, a series of P-Δ (load-displacement) curves are ploted and from these, the critical buckling load of the arches corresponding to each type of hangers system could be computed.

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Finally a comparaison between these load-displacement curves is made, which could give some informations about the influence of each type of hangers system on the general stability of this arch steel bridge.

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Particular aspects concerning the use of electronic computers into bridge design process

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Abstract

In bridge design process, a number of problems raise with regard to structural and behavior analysis. The high data volume to be analyzed and the financial restrictions that apply warrant the use of modeling and simulation procedures in effective decision making. The paper reviews some particular aspects concerning the use of electronic computers, statistical analysis and mathematical algorithms into bridge design process.

The present article is structured in the following chapters:

Introduction; ***DESIGN SOLUTION ANALYSIS AND STRUCTURE SIMULATION, MATERIAL'S BEHAVIOR SIMULATION, CONCLUSIONS.***

The paper proposes a new approach in informatics concept of the bridge engineering in concept. Bridge engineering and systems engineering are connected to form a scientific background where principles and methods proper to systems theory, reliability theory, and information theory are applied in the study of bridge engineering cycle's stages: design, erection, operation, and evaluation. As a result, the image of a complex system is proposed. This system involves two sub-systems: the bridge and the computer. The goal of this construction is to resolve the issues related to bridge behavior using the instruments offered by information technology.

In the second section, we analyzed the modes of structural design problems analyses including through simulation. The paper comments the design software developed on the bases of finite element method (FEM). Results obtained by using the FEM to analyze a bridge structure are also presented.

In bridge design one must know the behavior of the actual behavior of the materials in the structure, environmental conditions, chemical and physical aggressiveness of the air and water and their actions on the material. The article, analyzes a method based on simulation. As an example, the corrosion process is presented starting from the Fick's diffusion law. The analysis uses several probability distribution functions: deterministic, normal, log-normal, and Beta distribution.

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Rehabilitation of concrete bridges with the concrete over slab

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Abstract

Considering the bridges rehabilitation program that is presently taking place in our country, the bridge experts have analyzed and developed various methods of rehabilitation. Among these methods, the concrete over slab of old bridge superstructures is a preferable choice, because it allows the increase of the bearing strength of the superstructure and the increase of the bridge's width allowing in the same time construction under traffic.

An important issue that has to be solved, is assuring that the old superstructure and the concrete over slab work together in such a way that the slab will contribute to the increase of the bearing strength of the old structure. For this purpose, it is necessary to design connectors that are fixed with resin or cement slurry in the old superstructure. The dimensioning of connectors is done for the shear forces that occur at the interface between the old structure and the concrete over slab. These shear forces are the result of the actions that emerge on the structure after the hardening of the concrete over slab (the weights of the bridge carriageway and of the sidewalks and the loads occurring from vehicles and people).

In this paper, is presented the method of determining the area of the connectors, in two hypothesis: one that consideres that the concrete has no contribution in assuming the shear forces that develop on the contact surface, and the other one that stipulates that the adherence between the two surfaces plays an important role (according to the Romanian design standards). On the other hand, the regulations of the "EN 1992-1, Eurocode 2– Part 1" are presented, focusing on the rules concerning the working together of two concrete elements of different ages linked by connectors.

Solutions for structures as continuous girders or frames and cantilever slabs, where the risk of shear displacements between the concrete over slab and the old superstructure occur, are also presented.