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PARAMETER ESTIMATION AND FE MODEL UPDATING OF A FULL SCALE BRIDGE ASSITED BY PARIS SOFTWARE

BY

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Abstract. This paper aims to present the benefits of an automated finite element model updating software, entitled PARIS PARameter Identification System in the field of Structural Health Monitoring SHM using nondestructive test NDT. The PARIS software shows promising applications for parameter estimation methods based on finite element FE models for bridge structures, by using static and modal measurements, as input data, for the estimation of stiffness and mass parameters at the element level of the bridge. The resulting updated model can be useful in the process of further validation of a simulated damage test data. Two validation examples using simulated nondestructive test data for updating a full-scale bridge model situated in Iaşi municipality under a typical damage scenario for bridges is presented in this paper. The results of the model updating process are then presented in order to validate the feasibility of using static nondestructive test data for the successful stiffness parameter estimation for the selected damaged elements of the bridge model. The modal response of the updated models is further compared to the initial undamaged model, in order to depict the influence of the simulated damage scenarios on the dynamic characteristics of the bridge structure.

Keywords: finite element model updating; parameter estimation; structural health monitoring.

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1. Introduction

In Europe, most of the bridges have exceeded their designed life span, as they were built up during the post second world war period. Nowadays, these structures are experiencing a loss in performance due to the presence of aggressive agents. The decrease of the constitutive materials durability has a direct effect of the serviceability of the bridge, (Richard *et al*., 2010). Road bridges begin to deteriorate once they are built and used, and thus the task of maintaining these structures safe and reliable for daily use is of great importance, (Alvandi & Cremona, 2006). Road bridges may also experience severe deterioration due to natural hazards, ageing, and increased structural performance demands that increase over time. Bridges are part of a transportation infrastructure network and play a major role in the urban and economic development and quality of life. Bridges that are unavailable due to maintenance and repair actions may have a negative social and economic impact. The partial or total closure of these critical components of the transportation network can result in major disruption such as long diversions, additional congestion, (Orcesi & Cremona, 2010).

Three main reasons can be pointed out, for assessing or evaluating the condition of existing bridges, (Richard *et a*l., 2010):

1. The general increase in traffic weights, increased traffic densities require bridges to carry greater traffic loads than those for which it was designed for;
2. Decrease in strength due to the deterioration or substantial damage to the bridge structure;
3. Changes in design codes that reduce the acceptable safety levels.

One aim of the assessment process is to establish a safe load carrying capacity for the existing bridges. In the last decades, the traffic loads and speeds have drastically increased and as a consequence, many existing road bridges are now subjected to loads and speeds far higher than those for which they have been designed for. Moreover, the lack of a reliable maintenance framework, has contributed to the severely deterioration of the bridges over their years of service. In Europe a study entitled *Sustainable Bridges* was performed in order to determine the impact of the increase of the transport capacity in the service life of existing bridges. The collected data from this project was used to check if applied analytical and numerical models are correctly representing the structural behavior of the existing damaged bridges, (Richard *et al*., 2010).

The topic of maintaining safe and reliable bridges for daily use has received considerable attention in literature in recent years. Current damage detection methods rely on visual or localized experimental methods such as acoustic, ultrasonic, magnetic field and radiographic methods, (Alvandi *et al*., 2008). Traditional inspections carried out on bridges require the portion of the structure being inspected to be readily accessible, but these procedures often interfere with the operational conditions, (Alvandi & Cremona, 2006). These methods often require that the location of the damage to be known a priori and that the portion of the structure being inspected to be readily accessible. Due to these limitations, these methods detect the damage on or near the surface of the bridge. Currently there is no experimental method general enough to be applicable to all the different portions of a bridge due to their limitations.

There is an increasing need for global damage detection methods that over-come the above limitations of the standard used methods. Non-destructive techniques NDT can be viewed as the means by which structures may be inspected without disruption or impairment of its serviceability. Some NDT techniques are based on visual observations and some are based on comparing and analyzing the properties of the materials of the bridge. Another NDT technique is to interpret the structural condition of the bridge by observing the change in its global behavior. This can be achieved by the use of vibration test data.

These non-destructive and global techniques for bridge structures diagnosis have led to the development of various methods for examining the changes of the dynamic characteristics of the bridge. These approaches have already been used for years now and are still being used in fields such as automotive, aeronautical and mechanical engineering, (Alvandi & Cremona, 2006). The global methods can be applied to more complex bridge structures. In the global damage detection methods, the modal parameters, natural frequencies and mode shapes, are considered functions of the physical properties of the bridge, mass, damping and stiffness. Therefore, variations in physical properties, such as in the stiffness or flexibility, will determine modifications in the modal properties. The most widely used modal properties in damage detection are eigenfrequencies and mode shapes. The variation in frequency and mode shape can be used to locate and quantify damage. These techniques that rely on frequency and mode shape variation are generally not very sensitive for local and moderate level of damages. The low sensitivity of these parameters requires in most cases either very precise measurement or large levels of damage in the structure of the bridge, (Alvandi et al., 2008).

Vibration-based damage identification techniques (VBDIT) is a NDT method that comprises in the application of an excitation force to the bridge followed by the interpretation of the response of the structure to the excitation. From recorded response of the bridge is then analyzed in order to extract the desired parameters, for instance, modal parameters, (Alvandi & Cremona, 2006).

The damages detected in a bridge can lead to expensive maintenance measures and, in case of extreme hazardous events with significant magnitude, dramatic social and human consequences can result. An efficient maintenance strategy should be able to identifying damages in an early stage, which are generally associated to local phenomena of small magnitude.

Damage identification for road bridge systems can be performed by using model based or data driven approaches. The first type aims to identifying damage by fitting a numerical model to real data, followed by optimization techniques. Conversely, the data driven approaches are based on data processing from in situ monitoring. This latter approach does not rely on a priori models.

Damage detection can be described as a four-level scale:

1. Damage detection;
2. Localization;
3. Type and severity assessment;
4. Lifetime prediction update.

The first and second levels can be carried out by using only data driven methods, while the fourth and partly the third stage level requires the use of numerical models. The last two levels may also require non-destructive testing, visual inspection, human expertise and additional theoretical concepts in order to enhance the damage detection process.

For the early damage detection approaches it's recommended to use damage sensitive feature extraction, since the acquired data from in situ measurements, alone, are not always informative about the presence of damage. Modal or modal-based quantities are the most used features for damage detection. Autoregressive models and wavelet components have also proven to be damage sensitive feature extractors for both static and dynamic monitoring, (Santos *et al*., 2013).

The lack of optimized maintenance strategies, greatly affects the structural safety of the existing bridges. Thus it becomes essential to find ways to preserve or even improve the existing bridges that are viable from an economical point of view, (Richard *et al*., 2010). The use of cost-effective maintenance strategies is crucial to the networks performance. One such bridge maintenance model, proposed by Liu & Frangopol (2005) uses a time-dependent bridge network reliability and bridge reliability importance factors to determine the importance of each bridge in the transportation network. The bridge network maintenance process is optimized by means of a stochastic dynamic program. Another model, proposed by Orcesi & Cremona (2011), uses a reliability-based network-level framework to optimize the maintenance strategies of the bridge network. This model enables the assessment of the expected maintenance and failure costs. The probability of equipment maintenance is also determined, (Orcesi & Cremona, 2010).

Orcesi & Cremona (2009) propose a Markov-chain framework to determine the overall condition of reinforced concrete bridges, that can help assess the funding required for the maintenance works. This framework has the following steps:

1. Propose a network-level approach where the lifetime-based indicator is determined from the visual inspection results, for each bridge of the network. The costs are assessed using a traffic assignment method that distributes the traffic volume among different routes within the transportation network. This method is very efficient in modeling complex networks.

2. Optimize the maintenance strategies of the bridge network by quantifying the performance of each bridge and the uncertainties that can appear in the future decisions process. An event tree that includes these uncertainties is built, based on the condition of each bridge. This event-tree helps predict all possible outcomes in the future inspections and also to calculate the expected maintenance and failure costs. A failure event is defined by the authors as the need to perform an unexpected rehabilitation of the bridge. The optimal solutions for the maintenance strategies are determined via a genetic algorithm.

3. Compare the optimal maintenance strategies that result from the previous steps by taking into account the interests of the users and the bridge owner.

2. General Description of the PARIS Software

Advances in technologies have led to the development of modern computational tools capable of analyzing large structural bridge systems in an efficient and accurate manner, that can be used in comprehensive infrastructural inspections and monitoring methods for road bridges.

PARIS (PARameter Identification System) is a custom MATLAB based computer program that integrates with the Finite Element Analysis (FEA) program, SAP2000, used for parameter estimation and FE model updating. The software developed by Sanayei (1997) uses static and modal measurements as input data, to estimate the parameters for stiffness and mass at the element level.

By observing the structure's global response from the NDT measurement data the program can be used to evaluate the structural health of the desired structural elements of a bridge. The program utilizes SAP2000 as the Finite Element Analysis (FEA) solver along with 3D model creation and validation. PARIS enables MATLAB that has great function optimization capabilities, to use SAP2000 as a slave program for FEA. The combination of these two powerful computational software platforms facilitates the development of an automated FE model updating computer program of full-scale structures (Sanayei & Rohela, 2014).

PARIS calibrates the FE model of a bridge by minimizing the residual between the predicted response of the FE model created in SAP2000 and the measured response from NDT data from in situ measurements.

The parameter estimation process starts with the creation of a FE model in SAP2000, based on the preliminary guess of the unknown structural parameters. These parameters typically are the rigidities and mass properties of the component elements. For the frame elements of a FE model the stiffness parameters are the axial rigidity, *EA*, bending rigidity, *EI* and torsional rigidity, *GJ*. For shell and solid elements, the stiffness parameter is represented by the modulus of elasticity, *E*, and for joint springs respectively the stiffness parameters are represented by the translational stiffness, k_X, and rotational stiffness, k_θ. The mass parameter is represented by the mass, *m*, of the finite elements. The changes in these structural parameters are used for defining the structural damage of the FE model.

2.1. Error Functions Used in the Parameter Estimation Process

Various error functions are used in the FE model updating process. The error functions are represented by the discrepancy between the predicted response of the FE model and the measured or simulated NDT data. These error functions are later used to create scalar objective functions that are optimized in MATLAB. The unknown parameter values are updated iteratively and a new response is calculated for the FE model, within each iteration step. The iterations stop when the behavior of the physical model closely resembles that of the real structure (Sanayei & Rohela, 2014).

The FE models used by PARIS are comprised of frame, quadrilateral shell, and cuboid solid elements using static and/or modal data. The FE models can be used to update joint spring and 2-node link element stiffness.

The error functions used by PARIS measure the residual between the predicted response from the FE model and experimental NDT data. They are formulated in terms of unknown parameters and they are used in the parameter estimation process. The error functions are expressed as the difference between the analytical and the measured physical quantities, as shown in eq. (1):

$$e(p) = q_{\text{predicted}} - q_{\text{measured}}, \tag{1}$$

where: *q* is the response quantity which can either be strain or translation and rotation of a FE element subjected to static loads or modal excitation.

The static and modal error functions are later divided in stiffness and flexibility based error functions. The stiffness based error functions are used to measure the residual between applied forces, while the flexibility based error functions measure the residual between the measured displacements.

1. Static stiffness (SS)

The static stiffness error function, developed by Sanayei & Onipede (1991) can be used to predicted the displacement response from applied static test loads. These predicted displacements together with the measured displacement response could be used to detect damage in the structures at the element level. The relationship for the static stiffness error function is written as eq. (2)2):

$$e_{\mathrm{SS}}(p) = \left(K_{aa} - K_{ab}K_{bb}^{-1}K_{ba}\right)u_a + K_{ab}K_{bb}^{-1}f_b - f_a\,, \tag{2}$$

where: u_a and u_b are the vectors of measured and unmeasured displacements; K_{aa}, K_{ab}, K_{ba} and K_{bb} are submatrices of the stiffness matrix; f_a and f_b are the applied load force vectors corresponding to the subsets *a* and *b* of degrees of freedom DOF.

2. Static flexibility (SF)

The static flexibility error function was developed by Sanayei *et al*. (1997) and is represented by the following relationship:

$$e_{\mathrm{SF}}(p) = \left(K_{aa} - K_{ab}K_{bb}^{-1}K_{ba}\right)^{-1}\left(f_a - K_{ab}K_{bb}^{-1}f_b\right) - u_a\,, \tag{3}$$

where: u_a is the vectors of measured and unmeasured displacements; K_{aa}, K_{ab}, K_{ba} and K_{bb} are submatrices of the stiffness matrix; f_a is the applied load force vectors corresponding to the subset vector *a*, of DOFs.

This error function is based on the inverse force–displacement relationship and it compares the predicted and the measured displacements.

3. Static strain (SSTR)

The static strain error function is defined as the difference between the predicted and the measured strains and is given by eq. (4)4), (Sanayei & Saletnik, 1996):

$$e_{\mathrm{SSTR}}(p) = B_a K^{-1} f_a - \varepsilon_a\,, \tag{4}$$

where: B_a is the mapping matrix; K is the stiffness matrix; f_a is the applied load force vectors; ε_a is the strain from displacements corresponding to the subset *a*, of DOFs.

For shell elements PARIS uses a more direct approach based on Hooke's Law, in which the strain calculated at a particular node is averaged based on the surrounding shell elements.

4. Modal stiffness (MS)

Sanayei et al. (1999) developed the modal stiffness error function and is represented by eq. (5)5):

$$e_{\mathrm{MS}}\left(p\right)_i=\Big[\left(K_{aa}-\lambda_i M_{aa}\right)-\left(K_{ab}-\lambda_i M_{ab}\right)\left(K_{bb}-\lambda_i M_{bb}\right)^{-1}\times \times\left(K_{ba}-\lambda_i M_{ba}\right)\Big]\phi_{ai}, \tag{5}$$

where K_{aa}, K_{ab}, K_{ba} and K_{bb} are submatrices of the stiffness matrix; M_{aa}, M_{ab}, M_{ba} and M_{bb} are submatrices of the mass matrix; ϕ_{ai} represents the measured modal displacements corresponding to the subset a of DOFs at ith iteration and λ_i is the square of the i^{th} natural frequency.

5. Modal flexibility (MF)

Sanayei *et al*. (2001) developed modal flexibility based error function by condensing the characteristic equation written in terms of the flexibility matrix. Hjelmstad (1996) also arrived at a similar modal flexibility based error function formulation by only partitioning the mass matrix at measured and unmeasured DOF without using condensation. Similar to static flexibility, the modal flexibility error function includes the inverse of stiffness matrix, K, in its formulation. Modal flexibility error function, as formulated by Sanayei *et al*. (2001), is shown in eq. (6)6):

$$e_{\mathrm{MF}}\left(p\right)_i=\Big[\lambda_i^2 D_{ab}\left(I-\lambda_i D_{bb}\right)^{-1} D_{ba}+\lambda_i D_{aa}-I\Big]\phi_{ai}, \tag{6}$$

where: D_{aa}, D_{ab}, D_{ba} and D_{bb} are submatrices of the dynamic matrix; I is the identity matrix; ϕ_{ai} represents the measured modal displacements corresponding to the subset a of DOFs at i^{th} iteration and λ_i is the square of the i^{th} natural frequency.

2.2. Validation of the Estimated Parameters

The difference in the order of magnitudes between the various types of parameters, measured quantities, and error functions used in PARIS for the estimation process can give rise to numerical difficulties. There are two main types of normalization methods, used within PARIS software: parameter normalization and error function normalization.

The normalization processes for the parameters that define the structural properties are made with respect to their initial values. Dividing the parameter values by the initial estimated values is called parameter normalization. When the estimated value is 0 this indicates complete damage.

The optimization routine stops when the convergence criteria are meat. The convergence criteria are established so as the dissimilarity's between the updated unknown parameter values and true values are reasonable low.

The sensors and instruments that collect the measurements during a NDT can give inaccurate reading due to either the defects in the manufacturing process or the faulty handling and installation of these devices. The influence of the measurement errors on the estimated parameters can be studied in PARIS by introducing uniformly or normally distributed errors in the simulated data. The simulated NDT data is contaminated with proportional and absolute errors, respectively (Sanayei & Rohela, 2014).

3. Evaluating the Modal Response of a R.C. Bridge Located in Iaşi Municipality

A full-scale FE model updating example using simulated NDT data is presented to illustrate the capabilities of the PARIS program. The FE model is a scale model of a reinforced concrete road bridge situated in Iaşi municipality, as shown in Fig. 1. The bridge is located in the Tudor Vladimirescu residential neighborhood, and it consists of a single span continuous superstructure, with a total length of 46 m. The superstructure is made up out of a concrete box girder deck with 3 traffic lanes, with a width of 13.2 m. The bridge deck is supported by 6 girders that are interconnected by end-span diaphragms as well as intermediate diaphragms at a uniform spacing of 4.75 m. The superstructure is supported by two concrete wall-type bents of a height of 4 m. The bridge was models in SAP2000 using frame and shell elements of concrete material, C 30/37, with the following material properties: density of 2,548.5 kg/m^3, modulus of elasticity E of 33,000 MPa, Poisson's ratio ν of 0.2, shear modulus G of 13,750 MPa, and the characteristic strength of concrete f_c of 30 MPa. The finite element model used to describe the bridge is made up out of 119 shell elements that make up the bridge deck, 76 frame elements that represent the girders that support the bridge deck, and 144 joint elements.

Fig. 1 – RC Bridge location in Iaşi Tudor Vladimirescu residential neighborhood.

PARIS was used to estimate the unknown stiffness for two simulated damage scenarios, see Table . In the first damage case, a crack situated at the middle span of the bridge was considered for all the girders supporting the deck. This scenario considers the cracks to be represented by a theoretical 100% reduction in axial rigidity EA for the FE element where the damage occurs. Error functions SS, SF, and SSTR were run individually in order to estimate the EA parameter of the damaged elements. In the simulation the crack was considered to by represented by a theoretical 100% reduction in bending capacity EIzz for the damaged elements. The unknown parameters were estimated using the SS, SF and SSTR error functions.

Diagnostic static test loads were applied under three load cases, corresponding to each error function, see Table 2. The location and magnitude of the loads was established according to STAS-3221 (1986), corresponding to V80 moving vehicle load class, as shown in Fig. 2.

Table 1
Damage Case Summary for Bridge Model

Damage case	Damage case description	Damaged element	Error function
1	100% axial rigidity EA loss	60,72,73,74,75,76	SS, SF, SSTR
2	100% bending rigidity EIzz loss	60,72,73,74,75,76	SS, SF, SSTR

Table 2
Applied Static Loads on Bridge Model.

Load case	Location of applied load	Direction	Load, [kN]
1	99, 101, 107, 109, 113, 116, 121, 123	*Z*	–100
1	1, 2, 3, 4, 26, 28, 30, 32, 83, 84, 85, 86	*Z*	–1,385

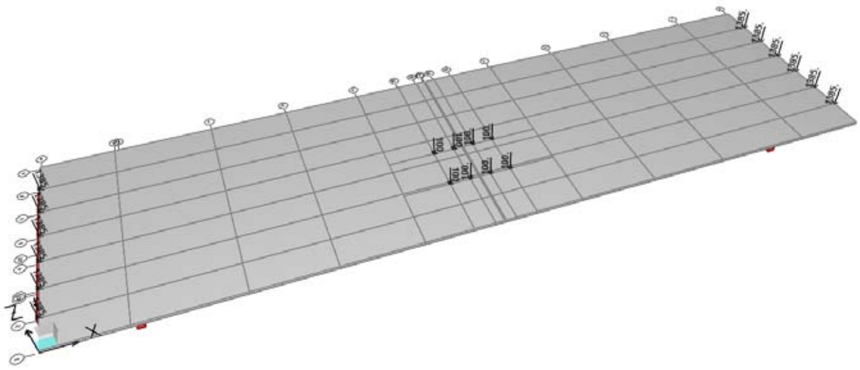

Fig. 2 – Finite Element model of bridge structure, and applied loads.

The strain measurement locations for static load cases necessary in for the SSTR error function estimation, used with the two damage cases are listed in Table 3.

Table 1
Strain Measurement Locations for Bridge Model

Damage case	Load case	Measured element location	Strain gauge SG location, [m]		
			X	*Y*	*Z*
1	LC1	25	2.75	–0.998	0
		27	2.25	–0.525	0
		31	2.25	–0.67	0
2	LC1	25	2.75	–0.998	0
		27	2.25	–0.525	0
		31	2.25	–0.67	0

The parameter estimation results for the first damage case are presented in Fig. 3, and for the second damage case in Fig. 4, respectively. The first column represents the normalized value of initial guess of the parameter. Column number 2 is the true value of the parameter used to simulate NDT data. The last column represents the parameter estimates using simulated measurements for error functions SS, SF, and SSTR, respectively. The SS, SF, and SSTR error functions were used independently and successfully in estimating the EA and EIzz parameters for damage case 1 and damage case 2 respectively, showing that the unknown axial rigidity and bending rigidity converged to values very close to the true values of the EA and EIzz that were used for the simulating NDT data.

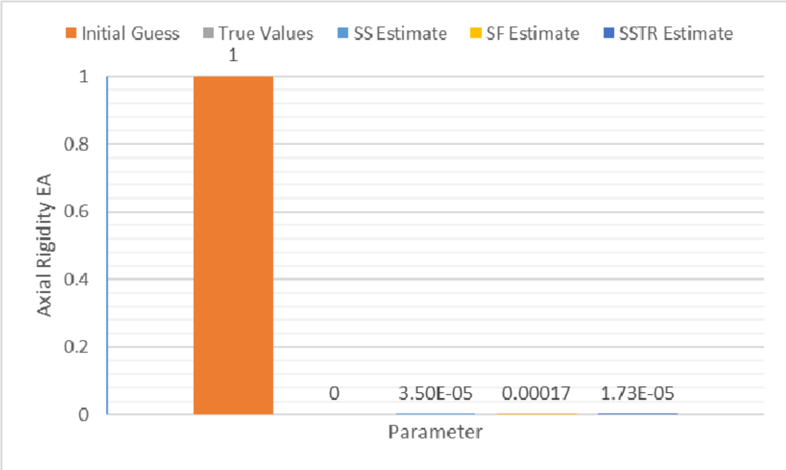

Fig. 2 – Damage Case 1: EA estimated parameters for the bridge model.

For the first damage scenario the unknown estimated values for the axial rigidity have relatively close values to the true value of EA used to represent the damage scenario. The SSTR error function has converged in this case closest to the initial true value.

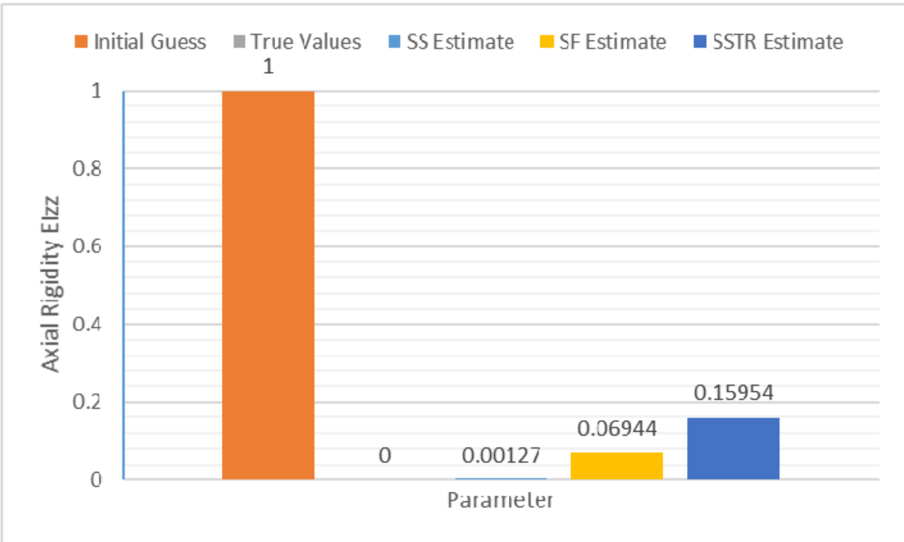

Fig. 4 – Damage Case 2: EIzz estimated parameters for the bridge model.

The second damage scenario show some variation between the estimated values for the unknown bending rigitity parameters. The closest values to the initial simulated damage are given by the SS error function.

The presented results of the model updating process show that PARIS has successfully used the nondestructive test data in detecting the damaged elements of the bridge model.

For each parameter estimation iteration PARIS has updated the FE model according to the simulated damage scenarios. From these updated models one can run further structural analysis in order to determine the dynamic characteristic of the damaged model and asses the effect of the damaged elements on its structural behavior. In Figs. 5 and 6 the variation of the fundamental period of vibration T corresponding to the undamaged model is represented alongside the periods on vibration for the damaged models subjected to first damage scenario and to the second damage scenario respectively.

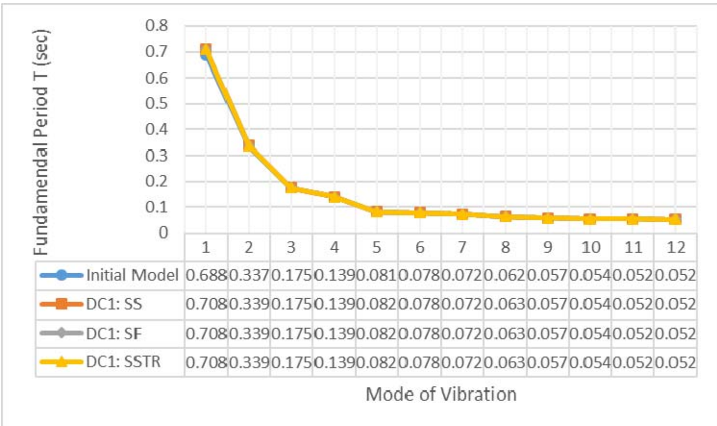

Fig. 5 – Damage Case 1: Modal response of the updated models.

	1	2	3	4	5	6	7	8	9	10	11	12
Initial Model	0.688	0.337	0.175	0.139	0.081	0.078	0.072	0.062	0.057	0.054	0.052	0.052
DC2: SS	0.696	0.339	0.175	0.139	0.082	0.079	0.073	0.062	0.057	0.054	0.052	0.052
DC2: SF	0.693	0.338	0.175	0.139	0.082	0.079	0.073	0.062	0.057	0.054	0.052	0.052
DC2: SSTR	0.691	0.338	0.175	0.139	0.082	0.079	0.072	0.062	0.057	0.054	0.052	0.052

Fig. 6 – Damage Case 2: Modal response of the updated models.

4. Conclusions

The studied examples, presented in this paper demonstrate that the application of PARIS software for parameter estimation and FE model updating enables researchers an advanced programming, based on optimization procedures provided by MATLAB and the analysis capabilities of structural analysis software SAP200.

The methodology presented in this case study is applicable for FE model updating of full-scale bridge structures, modeled with a large number of finite elements. PARIS software gives also the access to various mathematical functions, matrix operations, and optimization routines that are very useful in handling a larger database of parameter estimation problems.

This paper illustrates also the feasibility of using static and modal information, as nondestructive input data for the successful full-scale FE model's updating, allowing a substantial benefit in effective studies and thorough structural health monitoring of bridge constructions.

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ESTIMARE PARAMETRICĂ ŞI ACTUALIZAREA UNUI MODEL DE EF LA SCARĂ REALĂ PENTRU UN POD ASISTATĂ DE SOFTUL PARIS

(Rezumat)

Se prezintă un studiu de caz complex, evidenţiind beneficiile utilizării software PARIS *PARmeter Identification System* în domeniul de evaluare a podurilor de durata ciclului de viaţă, utilizând teste nedistructive NDT şi simulări în paralel efectuate cu programe de calcul de Element Finit, respectiv SAP2000. Utilizarea programului de calcul PARIS a condus la rezultate foarte promiţătoare privind estimare parametrică bazate pe modele cu element finit a comportării unor clase de structuri de poduri. Datele iniţiale de intrare au constat din date experimentale statice şi informaţii modale, utilizate în procedura de estimare/corectare a parametrilor de rigiditate si masă la nivelul de element al podului studiat. Modelul actualizat/corectat al structurii de pod , rezultat din estimare este necesar în procesele viitoare de validare a datelor experimentele pentru modele structurale cu degradări simulate pe durata ciclului de viaţă al podului. În acest articol se prezintă două exemple de validare a comportării unei structuri de pod, utilizând rezultatele unor simulări de teste nedistructive, necesare în procesul de actualizare- corectare a modelului de EF, pentru o structură integrală a podului Tudor Vladimirescu situat în judeţul Iaşi, expus prin simulări la o serie de scenarii de degradare tipice pentru structurile de poduri. Rezultatele procesului de actualizare, respectiv corectare a modelului de EF sunt utilizate în validarea, sau calibrarea rezultatelor înregistrărilor statice nedistructive, ce au permis ulterior estimarea parametrilor de rigiditate pentru elementele de pod degradate selectate. Răspunsul modal al modelelor corectate şi respectiv actualizate este comparat ulterior cu datele privind răspunsul modal obţinut pe modelul de EF iniţial, nedegradat. Metodologia de cercetare aplicată indică astfel influenţa deteriorărilor considerate în diferite scenarii simulate de degradare asupra caracteristicilor dinamice ale structurii de pod, constituind o baza de informaţii foarte utile în monitorizarea pe durata ciclului de viaţă a cestor tipuri de infrastructuri de construcţii.