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SYSTEM IDENTIFICATION OF A R/C BRIDGE BASED ON AMBIENT VIBRATIONS AND 3D NUMERICAL SIMULATIONS OF THE ENTIRE SOIL-STRUCTURE SYSTEM

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Abstract. *The scope of this paper is to identify the parameters affecting the dynamic response of an existing R/C bridge, based on low ambient amplitude vibration measurements and numerical predictions using complex finite element models. For this purpose, the instrumented, 2nd Kavala Bypass Ravine Bridge constructed along the Egnatia Motorway Greece is studied and a refined three- dimensional (3D) FEM is developed that takes into consideration the coupling and dynamic interaction of the overall superstructure-foundation-soil and deck-abutment-embankment system. The instrumentation schemes and the necessary algorithms applied for computing the modal characteristics of the bridge are discussed, while the modelling assumptions made for the soil-structure system are comparatively assessed and justified for various models of different levels of complexity. Given the large number of the system's degrees of freedom, a manual, modal-based FEM updating method is also presented. The results show good agreement between the measured and computationally predicted dynamic characteristics of the structure. They also show that the accurate estimation of the pier, deck and bearings stiffness is a key parameter for reliable system identification.*

1 INTRODUCTION

Structural health monitoring and identification of structural modes of existing civil engineering projects is nowadays a major component of system maintenance and management especially for highway transportation networks. The modal characteristics of structures can be evaluated either by identification systems that are based on ambient (i.e., due to traffic and wind) and earthquake-induced vibrations or by modal analysis of finite element models (FEMs). Important information can be derived by the comparative assessment between the results of the two methodologies as scientists can investigate the structural integrity of structures and also validate the reliability of the FEMs that are developed for the analysis, design and assessment of structures.

An identification system based on ambient vibrations involves the presence of factors imposing ambient vibrations to a structure, monitoring of the structural response (measured acceleration time histories) and the development of an appropriate mathematical algorithm. Such an algorithm uses the measured output data as an input for the prediction of the modal frequencies, modal damping ratios and mode shapes through classically damped or non-classically damped modal models**.** A significant number of methods [1] and the respective software have been developed for the identification of modal properties, based on ambient vibrations, both in the time and frequency domain. The methods, based on output measurements only, make the assumption that the input can be well represented by a vector white noise process. Recent developments are also reported in Peeters and De Roeck [2] and Basseville et al. [3] using time domain stochastic subspace identification methods, in Beck et al. [4] using time domain least-squares methods based on correlation functions of the output time histories, in Verboven [5], Gauberghe [6] and Brincker et al. [7] using frequency domain least-squares methods based on full cross-power spectral densities (CPSD), and in Peeters and Van der Auweraer [8] based on half spectra. Bayesian and maximum likelihood statistical methods have also been proposed, for example, in Katafygiotis and Yuen [9], Guillaume et al. [10] and Verboven [5].

On the other hand, the estimation of dynamic characteristics using modal-based analysis of finite element models requires appropriate software to carry out dynamic analysis. However, in the case of extended structures such as bridges, the compliance and damping of the supporting soil has to be taken into consideration as it may affect significantly both the dynamic characteristics of the soil-foundation-superstructure and the embankment-abutment-superstructure system. An efficient way to account for this phenomenon is to de-couple the problem into a kinematic and an inertial sub-structure (Mylonakis and Gazetas [11]). Alternatively, a holistic numerically modelling of the entire soil, structure and foundation system is also feasible (i.e. Wolf [12]).

Despite above advances, it is still quite common to observe differences between the measured dynamic characteristics of the structure and the dynamic characteristics predicted numerically by FEMs. For that reason, mathematical algorithms have been developed based on the identified modal characteristics (e.g. Mottershead and Friswell [13], Bohle and Fritzen [14], Teughels et al. [15], Lam et al. [16], Christodoulou and Papadimitriou [17]) that permit the re-estimation of the structural parameters of the finite element models and the minimization of the induced error. These algorithms use the identified modal data and formulate them as weighted least-squares problems in which the optimal values of the structural parameters of a FEM are obtained by minimizing a measure of the residuals between the measured and numerically predicted modal characteristics. Alternatively, the structural parameters of the finite element model can be automatically updated, based on the identified modal data, without the development of a mathematical algorithm. This automatic model updating procedure solely consists of a sequential parametric analysis for different values of the structural parameters, until the percentage of error between the measured and predicted modes is minimized. Both the aforementioned calibration procedures for model updating provide an insight into the epistemic uncertainties related to the simplified assumptions and idealizations inevitably made during the development of FEMs. These uncertainties include the uncertainty in the model topology, the uncertainty in the boundary conditions of the model and the uncertainty in the material properties.

Along these lines, the scope of this paper is to qualitatively and quantitatively validate the uncertainties associated with the modeling assumptions made and to estimate which of them influences most the reliability of the results, at least for the particular bridge studied. For that purpose, a modal identification approach based on ambient vibrations, three FEMs of different modeling complexity and two model updating methods (automatic and manual) were implemented. The description of the bridge, the methodology adopted and the comparative assessment of the results obtained are presented in the following.

2 DESCRITION OF THE BRIDGE STUDIED

2.1 Structural system

The 2nd Kavala Bypass Ravine Bridge, shown in Figure 1, is a newly built bridge located in Section 13.7 of Egnatia, a major 670km highway constructed on the traces of the ancient Roman path, crossing northern Greece from its western to its eastern border. Its overall length is 170m and comprises two statically independent branches, with four identical simply supported spans of 42.5m. Each span is built with four precast post-tensioned I-beams of 2.80m height that support a continuous deck of 26cm thickness and 13m width. The I-beams are supported by 2 abutments (A1 and A2) and by 3 piers (M1, M2 and M3) through laminated elastomeric bearings. Each abutment has 4 cyclic bearings and each pier has 8 rectangular bearings. The piers have a 4×4m hollow cross-section, 40cm wall thickness and heights equal to 30m (M1, M3) and 50m (M2), all supported with large caissons on relatively stiff soil (corresponding to soil class “A” according to both the Greek Seismic Code and the Eurocode 8 soil classification). The four spans of the deck are interconnected through a 2-m long 20-cm thick continuity slab over the piers.

2.2 Instrumentation

Between the two identical branches of the 2nd Kavala Bypass Ravine Bridge only the southern branch is instrumented. The instrumentation consists of a 24-accelerometer array [1], one at each deck side and located at the middle of each bridge span. More specifically, the accelerometers are installed both on the external sidewalk of the deck and on the internal New Jersey barrier of the deck. As shown in Figure 2, 18 of the 24 sensors are installed on the deck and two at the top of each of the three piers (six in all) next to the laminated elastomeric bearings so that adequate information is provided to distinguish between the pier and bearing stiffness. The sensors on the deck have a 3-letter label that follow the above explained convention: The last letter denotes the orientation of the uniaxial sensor (L: longitudinal, T: transverse, V: vertical). The previous one denotes the side of the bridge deck on which the sensor lies (R: right, L: left). Finally, the first letter denotes the bridge section that the sensor lays on (A1, B1, C1 or D1). The numbers next to each sensor label denotes the length of the cable used to connect the sensor to each recording unit. The sensors on the piers follow the same convention used for the sensors on the deck with the exception that the letters U1, U2 and U3 refer to the top of the piers. The particular layout of the instrumentation permits the recording of the dynamic response of the bridge under ambient loads.

This recording system has start/common trigger capabilities to enable synchronous data acquisition. The trigger threshold can be set independently for each sensor, and the user can define the sensors that will cause a system trigger. The systems are equipped with GPS boards as well as with external GSM/GPRS cellular modems that allowed telematic control and data transfer to the user offices.

Figure 1: General overviews of the 2nd Kavala Ravine Bypas Bridge.

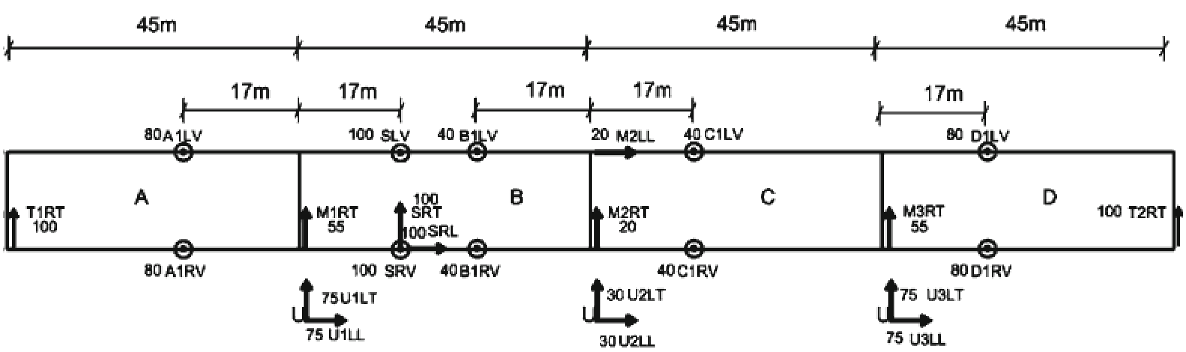

Figure 2: Instrumentation of the 2nd Kavala Ravine Bypas Bridge [1].

3 IDENTIFICATION OF STRUCTURAL MODES VIA AMBIENT VIBRATIONS

The methodology of identifying structural modes via ambient vibrations is based [1] on a least squares minimization of the measure of fit between the cross power spectral density (CPSD) matrix $\hat{S}(k\Delta\omega)\in C^{N_0\times N_0}$ and the CPSD matrix $S(k\Delta\omega;\psi)\in C^{N_0\times N_0}$. The $\hat{S}(k\Delta\omega)$ matrix is estimated from the measured output acceleration time histories and the $S(k\Delta\omega;\psi)$ matrix is predicted by a modal model. In Equation 1 N_0 is the number of measured degrees of freedom (DOF), $\Delta\omega$ is the discretization step in the frequency domain, $k = \{1, \ldots, N\omega\}$ is the index set corresponding to frequency values $\omega = k\Delta\omega$, $N\omega$ is the number of data in the indexed set, and ψ is the parameter set to be estimated.

$$E(\psi)=\sum_{k=1}^{N_\omega} tr\left[S(k\Delta\omega;\psi)-\hat{S}(k\Delta\omega)^{*T}\left(S(k\Delta\omega;\psi)-\hat{S}(k\Delta\omega)\right)\right] \quad [1]$$

Gauberghe [6], as described in Equation 2, gives the CPSD matrix $S(k\Delta\omega;\psi)$ with the assumption of general non-classically damped modes. In Equation 2 m is the number of contributing modes in the frequency range of interest, λ_r is equal to $(-\zeta_r\omega_r \pm j\omega_r\_1 - \zeta^2_r)$ and is the complex eigenvalue of the r-th contributing mode. In Equation 2 ω_r is the r–th modal frequency, ζ_r is the r–th modal damping ratio, $\varphi_r\in C^{N_0\times N_0}$ is the complex mode shape of the r-th mode, $A\in C^{N_0\times N_0}$, $B\in C^{N_0\times N_0}$ are real symmetric matrices accounting for the contribution of the out-of-bound modes to the selected frequency range of interest, and g_r $C^{N_0\times N_0}$ are vector quantities that depend on the characteristics of the modal model and the CPSD of the white noise input vector, while the symbol u^* denotes the complex conjugate of a complex number u.

$$S(\omega;\psi)=\sum_{r=1}^{m}\left[\frac{\phi_r g_r^T}{(j\omega)-\lambda_r}+\frac{\phi_r^* g_r^{*T}}{(j\omega)-\lambda_r^*}+\frac{g_r\phi_r^T}{-(j\omega)-\lambda_r}+\frac{g_r^*\phi_r^{*T}}{-(j\omega)-\lambda_r^*}\right] \quad [2]$$

$$+\frac{1}{(j\omega)^4}A+B$$

The CPSD matrix in Equation 2 is defined by the parameters ω_r, ζ_r, φ_r, g_r, $r = 1, \ldots, m$, A and B. The modal parameter set ψ contains the above mentioned parameters and has to be identified. For non-classically damped modal models, the total number of parameters is $2m(1 + 2N_0) + N^2_0 + N_0$.

The objective is the minimization of Equation 1 and that can be carried out efficiently [18], significantly reducing computational cost, by recognizing that the error in Equation 1 is quadratic with respect to the complex modeshapes φ_r and the elements in the matrices A and B. This observation is used to develop explicit expressions that relate the parameters φ_r , A and B to the vectors g_r, the modal frequencies ω_r and the damping ratios ζ_r, so that the number of parameters involved in the optimization is reduced from $2m(1 + 2N_0) + N^2_0 + N_0$ to $2m (N_0 + 1)$. This reduction is considerable for a relatively large number of measurement points. Applying the optimality conditions in Equation 1 with respect to the components of φ_r, A and B, a linear system of equations results for obtaining φ_r, A and B with respect to the g_r, ω_r and ζ_r, $r = 1, \ldots, m$. The resulting nonlinear optimization problem with respect to the remaining variables g_r, ω_r and ζ_r, where $r = 1, \ldots, m$, is solved in Matlab using available gradient-based optimisation algorithms.

The starting values required in the optimization are obtained from a two-step approach as follows. In the first step, conventional least squares complex frequency algorithms [5] are employed, along with stabilization diagrams, to obtain estimates of the modal frequencies ω_r and modal damping ratios ζ_r and distinguish between the physical and the mathematical modes. These values in most cases are very close to the optimal values. In the second step, given the values of ω_r and ζ_r, the values of the residue matrices $R_r = \varphi_r g T_r \in C^{N_0 \times N_0}$ in Equation 2 are obtained by first recognizing that the objective in Equation 1 is quadratic with respect to R_r, A and B, then formulating and solving the resulting linear system of equations for R_r, A and B, and finally applying singular value decomposition to obtain estimates of φ_r and g_r from R_r. Usually, this two-step approach gives results that are very close to the optimal estimates. However, for closely spaced and overlapping modes it is often recommended to solve the original nonlinear optimization problem with respect to g_r, ω_r and ζ_r, $r = 1, . . . ,m$, using the estimates of the two-step approach as starting values.

4 ALTERNATIVE FINITE ELEMENT MODELS DEVELOPED

Three FEMs of increasing modeling complexity were created based on the exact geometrical and material properties that were used for design. The first model is an one-dimensional (i.e., frame type), fixed-base model (hereafter called “1D-Fixed”), the second is a three-dimensional, fixed-base (“3D-Fixed”) and the third is a model for which the whole soil-foundation-structure system has been simulated in 3D space (“3D-3D Soil”). The numerical simulation of the 1D-Fixed model was carried out with the computer program COMSOL 2005 multi-physics, while the numerical simulation of the other two 3D models was carried out with ABAQUS 6.8.

The 1D-Fixed model of the Kavala bridge [1] was simulated using three-dimensional, two node, beam-type finite elements for the modeling of the structural elements (deck, I-beams, piers and bearings). This model is shown in Figure 3 and has 900 degrees of freedom. The cross-sectional parameters of each one of the longitudinal beam elements are those of an equivalent cross-section that accounts for the section of the post-tensioned beam, as well as the corresponding effective width of the deck plate. The transverse beams at the two ends of the span correspond to the existing cross-beams above the bearings, whereas the other four transverse beams represent the coupling of the longitudinal beams in the transverse direction due to the presence of the deck. Adjacent spans are interconnected with a 20-cm thick 2-m long plate. The piers and the abutments bearing are assumed fixed at their base.

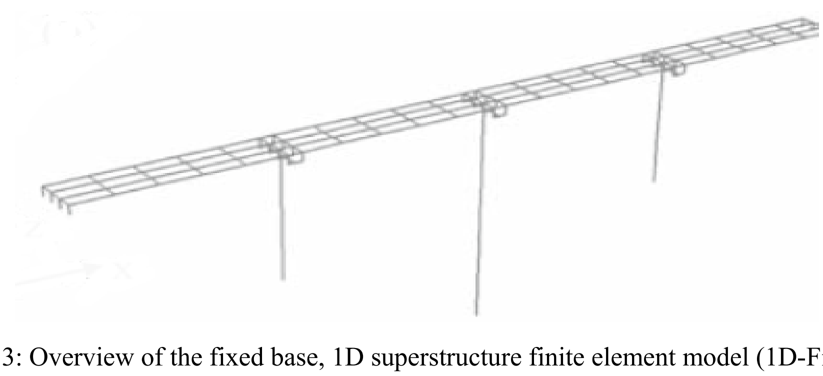

Figure 3: Overview of the fixed base, 1D superstructure finite element model (1D-Fixed).

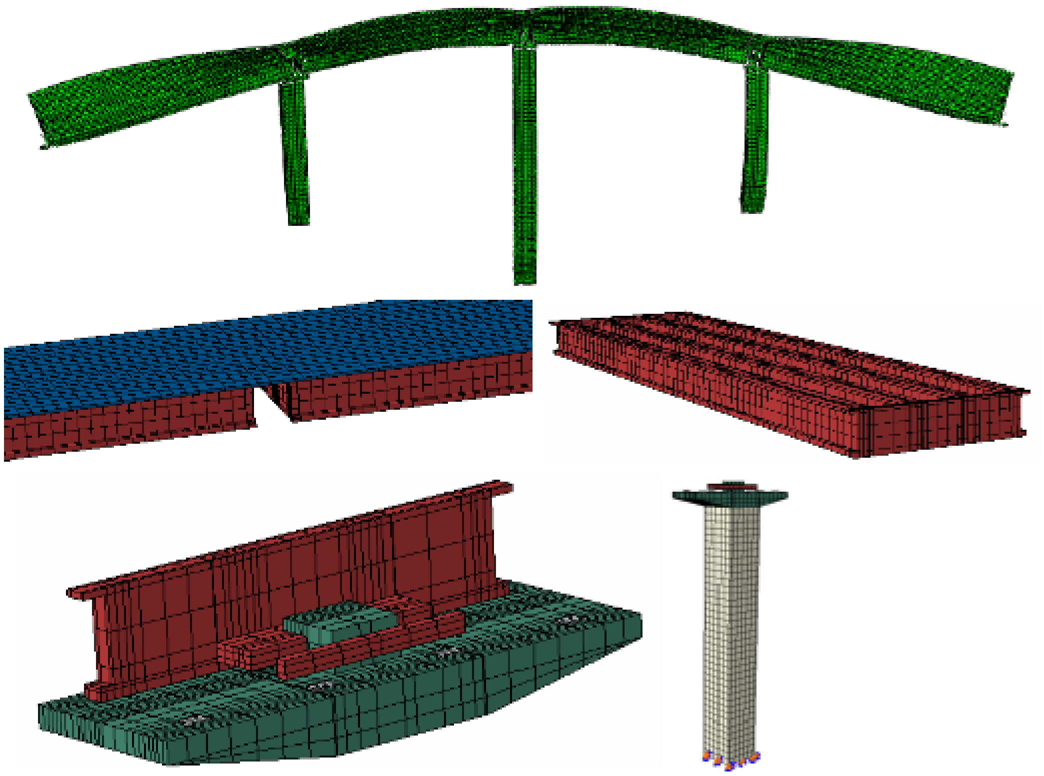

Figure 4: Overview and fundamental mode in the transverse direction (top) of the fixed base, 3D superstructure finite element model (3D-Fixed), as well as modelling details of the deck, stoppers and bearings (bottom).

The 3D-Fixed model of the Kavala bridge was simulated using three-dimensional eight node brick-type finite elements to model the entire superstructure. This model is shown in Figure 4 and has approximately 241,000 degrees of freedom. The mesh size of the elements used for bearing modeling was 0.25x0.25m while an average mesh size for the concrete sections was 0.75x0.75m. In this refined 3D FEM the deck, the I-beams, the piers, the bearings and the stoppers were modeled in maximum detail in 3D space. The abutments are considered as non-deformable, whereas the piers are assumed to be rigidly connected to the foundation, ignoring, in this version of the model, soil-structure interaction effects. Similarly, the piers and the abutment bearings are assumed fixed at their base.

The 3D-3D Soil model of the Kavala bridge is shown in Figure 5 and has approximately 960,000 degrees of freedom. In a similar fashion to the previous model, the superstructure is modeled in maximum possible detail. In addition, the entire soil-foundation-superstructure system is simulated, considering the exact geometry of the abutment-backfill-embankment system and the middle piers-caisson-soil substructure system. Mesh size for the superstructure was taken identical to the previous 3D-Fixed model and was also set equal to 2x2m for the surrounding and supporting soil. In order to reduce the computational time required, an additional, equivalent model of the 3D-3D Soil was also developed, after establishing a level of agreement between the two models by ensuring identical dynamic characteristics. This model is shown in Figure 6, 407,000 degrees of freedom and considered a smaller, though, adequate part of the soil volume. Given the compatibility of the two latter models in terms of system stiffness and damping, most analyses cases were carried out with this model instead of the initial 3D-3D Soil model.

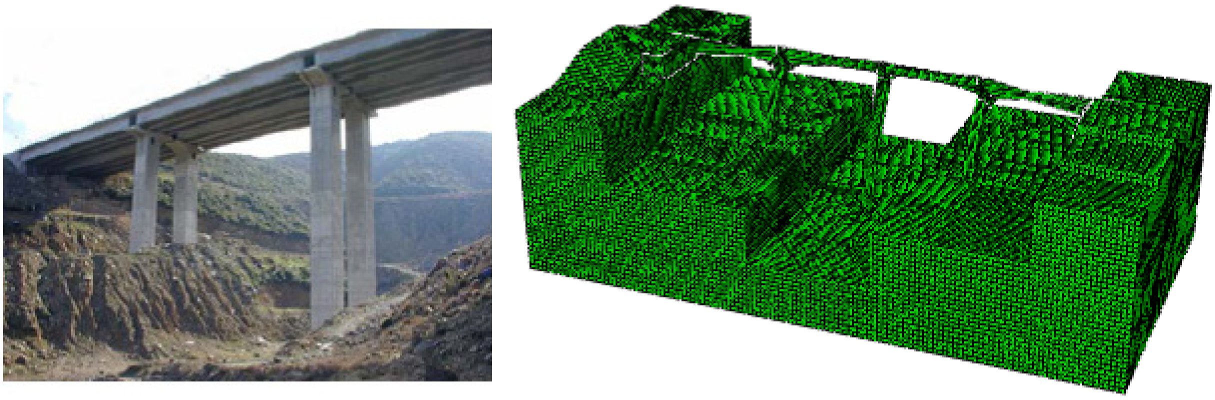

Figure 5: Overview and fundamental mode in the transverse direction of the 3D soil-foundation-superstructure finite element model (3D-3D Soil).

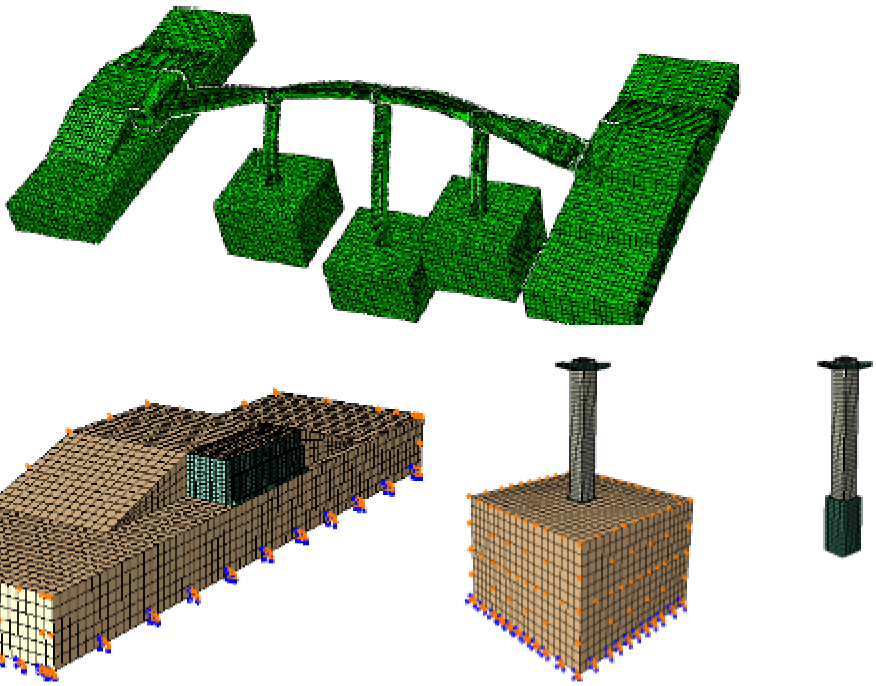

Figure 6: Overview and fundamental mode in the transverse direction of the equivalent 3D-3D Soil model as well as modelling details of the abutment-embankment system and the pier foundation subsoil.

5 FINITE ELEMENT MODEL UPDATING METHODOLOGY

The objective in a FEM updating methodology is to estimate the values of the structural parameter set $\theta \in R^{N_\theta}$ of a class of linear FEMs so that the modal frequencies and modeshapes $\{\omega_r(\theta),\ \varphi_r(\theta) \in R^{N_0},\ r = 1, \ldots, m\}$ predicted by the linear class of models best matches, in some sense, the experimentally obtained modal data $\{\hat{\omega}_r,\ \hat{\varphi}_r \in R^{N_0}, r = 1, \ldots, m\}$ contained in the set ψ, where m is the number of observed modes, and N0 is the number of recorded DOFs. The optimal values of the parameter set θ are obtained by minimizing the weighted modal residuals [17].

$$J(\theta;w)=\sum_{r=1}^{m}\left[w_{w_r}\frac{\left[w_r(\theta)-\hat{w}_r\right]^2}{\left[\hat{w}_r\right]^2}+w_{\phi_r}\frac{\left\|\beta_r\phi_r(\theta)-\hat{\phi}_r\right\|^2}{\left\|\hat{\phi}_r\right\|^2}\right] \quad [3]$$

The first norm in Equation 3 represents the difference between the measured and the model predicted frequency for the r-th mode, while the second norm represents the difference between the measured and the model predicted modeshape components for the r-th mode, where $\beta_r = \hat{\varphi}_r^T \varphi_r(\theta) / \varphi_r^T(\theta)\varphi_r(\theta)$ is a normalizing scalar guaranteeing that the measured $\hat{\varphi}_r$ is closest to $\varphi_r(\theta)$ for given θ. The weighting factors $w_{\omega r} \geq 0$ and $w_{\varphi r} \geq 0$, r = 1, . . . ,m, satisfy the condition $\Sigma^m{}_r$=1[$w_{\omega r}$+ $w_{\varphi r}$] = 1. The objective function J (θ;w) represents an overall measure of fit between the measured and the model predicted modal characteristics. Herein, conventional weighted least squares methods are used which assume equal weight values. Finally, the optimization of J (θ;w) in Equation 3with respect to θ can readily be carried out numerically using any available gradient-based algorithms for optimizing a nonlinear function of several variables. This procedure is described more detailed in Ntotsios et al. [19].

The above mentioned mathematical algorithm was applied for the updating of the 1D-Fixed model of the Kavala bridge. For that purpose, a Matlab code was developed in interaction with the computer program COMSOL, wherein the structure was simulated. In contrast, given the large number of degrees of freedom of the 3D-Fixed model and the 3D-3D Soil model of the Kavala bridge, their model updating was performed manually through a sequential parametric analysis scheme. The concept behind this simplified, manual updating approach for the 3D finite element models is described in detail in Section 6.

6 COMPARATIVE ASSESMENT OF THE PREDICTED AND MEASURED RESPONSE

6.1 Measured and predicted structural modes of Kavala Bridge

The accelerometers installed along the deck and at the top of the three piers recorded the bridge's response and the acceleration time histories were then used as input to the model updating mathematical algorithm described in Section 3. Through this procedure, two transverse, one longitudinal and four bending modes were identified, whose modal frequencies are summarized in Table 1. Since the scope of the present research work was to validate, both qualitatively and quantitatively, the modeling assumptions made and to identify their relative impact on the numerically predicted structural response, the three finite element models developed were assessed comparatively.

The first comparison was made between the 1D-Fixed and 3D-Fixed models and aimed to identify the differences that arise by the inherent simplifications of the one dimensional modeling in contrast to the refined three dimensional modeling of the piers, I-beams, bearings and stoppers. For this reason, the soil compliance was deliberately not accounted for. The results of the predicted modal frequencies from the above models (i.e., 1D-Fixed and 3D-Fixed) are shown in Table 1. It can be seen that the agreement between the two models was very good (it is only the first two bending modes where the difference exceeds 10%), a fact that reveals that the assumptions made for the simplified 1D-Fixed model were reasonable. Despite their agreement though, it was evident that both models fail to predict well the actual, measured response as they exhibit large deviations from the identified modal frequencies that exceed 55% in the longitudinal direction and 34% and 58% respectively for the first two modes, in the transverse direction. In general, it is observed that the modes measured via ambient vibrations are on average 32% higher than those predicted by the two finite element modes, thus, the real structure is identified as significantly stiffer than predicted using the 1D and 3D fixed-base models.

A second comparison was made between the 3D-Fixed and the 3D-3D Soil models in order to quantify the importance of soil compliance on the predicted dynamic characteristics of the structure. The results of this analysis are shown in Table 2. It has to be noted herein that in case that the soil volume is modeled along with the superstructure, the identification of the modal frequencies of the entire soil-structure system is not as straightforward as it is in the case of a fixed structure, due to the enormous amount of the vibrating soil volume and the strong coupling among the structural and the soil modes. As a result, the characterization of modes as “transversal” or “bending” has to be made very carefully on the basis of both modal participation factors and modal shape visualization.

In terms of the predicted modal frequencies (Table 2), as anticipated, the refined consideration of abutment-backfill-embankment and pier-foundation-soil flexibility leads to lower values of modal frequencies. In particular, the 1st longitudinal mode is found 10% more flexible while the reduction of bending modal frequencies varies between 20-30%. However, compared to the identified modal frequencies, the particular 3D-3D soil model leads to an even more flexible (36% on average) prediction of structural response. This result clearly indicates that model refinement and soil-structure interaction simulation alone, cannot guarantee reliable representation of system stiffness unless appropriate model updating is performed in advance.

	AV	1D-Fixed		3D-Fixed	
Modes	ω(Hz)	ω(Hz)	Δω(%)	ω(Hz)	Δω(%)
1st Transverse	0.81	0.53	34.57	0.52	35.80
1st Longitudinal	1.29	0.57	55.81	0.56	56.59
2nd Transverse	1.61	0.67	58.39	0.67	58.39
1st Bending	3.40	2.78	18.24	2.69	20.88
2nd Bending	3.46	2.82	18.50	2.75	20.52
3rd Bending	3.47	2.82	18.73	2.84	18.16
4th Bending	3.51	2.83	19.37	2.87	18.23
Average Δω(%)			31.94		32.65

Table 1: Modal frequencies identified via Ambient Vibrations (AV) and numerical modeling (1D-Fixed vs. 3D-Fixed) and respective error (Δω%).

	AV	3D-Fixed		3D-3D Soil	
Modes	ω(Hz)	ω(Hz)	Δω(%)	ω(Hz)	Δω(%)
1st Transverse	0.81	0.52	35.80	0.49	39.51
1st Longitudinal	1.29	0.56	56.59	0.54	58.14
2nd Transverse	1.61	0.67	58.39	0.65	59.63
1st Bending	3.40	2.69	20.88	2.48	27.06
2nd Bending	3.46	2.75	20.52	2.55	26.30
3rd Bending	3.47	2.84	18.16	2.71	21.90
4th Bending	3.51	2.87	18.23	2.73	22.22
Average Δω(%)			32.65		36.39

Table 2: Modal frequencies identified via Ambient Vibrations (AV) and numerical modeling (3D-Fixed vs. 3D-3D soil) and respective error (Δω%).

Structural parameter	Model Updating Method
	Algorithmic
	Model Updating Case 1
E bearings	9.07
E deck	1.57
E piers	1.63

Table 3: Modal updating results for the 1D-Fixed FEM through a proposed mathematical algorithm.

	AV	1D-Fixed M.U. Case 1		3D-Fixed M.U. Case 1	
Modes	ω(Hz)	ω(Hz)	Δω(%)	ω(Hz)	Δω(%)
1st Transverse	0.81	0.86	-6.17	0.84	-3.70
1st Longitudinal	1.29	1.24	+3.88	1.19	+7.75
2nd Transverse	1.61	1.44	+10.56	1.55	+3.73
1st Bending	3.40	3.52	-3.53	3.62	-6.47
2nd Bending	3.46	3.56	-2.89	3.65	-5.49
3rd Bending	3.47	3.60	-3.75	3.73	-7.49
4th Bending	3.51	3.61	-2.85	3.80	-8.26
Average Δω(%)			+4.80		+6.13

Table 4: Modal frequencies identified via Ambient Vibrations (AV) and predicted by updated 1D-Fixed and 3D-Fixed FEMs for Case 1, as well as their in between percentage of error Δ(ω).

6.2 Finite element model updating

As mentioned earlier, for low amplitude vibrations, the structure was found to be much stiffer as one could numerically predict, regardless of model refinement. Given the fact that the finite element model used was indeed as complex and refined as possible, the model-induced (i.e., epistemic) uncertainty can be deemed as relatively low. As a result, the deviations between the identified and numerically predicted modal frequencies can be attributed primarily to the uncertainty in the material properties, which seem to be a key parameter for the reliable estimation of the dynamic characteristics of the structure.

Initially, the updating of model 1D-Fixed was carried out according to the procedure outlined in Section 5. For the case of the 3D models though, this procedure was not feasible for two main prohibitive reasons; due to the large number of the degrees of freedom of the soil-structure system but also because there was no communication protocol between the software ABAQUS used for the 3D simulations and the Matlab program running the model updating algorithm. For that reason, a different strategy was developed the manual updating of the 3D models towards the identification of the nearly-optimal fit between the identified and numerically predicted response. The workflow of this strategy is illustrated in Figure 7 and can be summarized into the following steps:

(a) Automatic model updating was performed for the case of the 1D-Fixed model. The results of the updated structural properties [1] of the 1D-Fixed model are presented in Table 3 (hereafter called “model updating Case 1”.

(b) The above identified properties were used as the first, best guess for the case of the 3D-Fixed model. Table 4 shows that the average percentage of error between the measured and the predicted modal frequencies has decreased from 31.94% to 4.8% for the 1D-Fixed model and from 32.65% to 6.13% for the 3D-Fixed verifying the trend of manual model updating was correct. It was also observed that both models predict higher values of frequencies to all modes compared with the ones identified via ambient vibrations, with the exception of the 1st longitudinal and the 2nd transverse mode.

(c) Given the refinement of the 3D-3D soil model and its capabilities to consider superstructure-foundation-soil and deck-abutment-embankment interaction, it was evident that the model updating factors predicted in step (b) would not necessarily be valid. For this reason, the 3D-3D Soil model was only initially updated based on the "Case 1" combination of structural parameters (Table 5). It can be seen that the average percentage of error between the measured and the predicted modal frequencies was decreased from 36.39% to 5.55%. In order to improve the convergence, sequential parametric analysis was conducted. The idea was to gradually modify specific structural parameters through a step-by-step parametric analysis scheme, until a nearly-optimal fit was achieved (Figure 7).

	AV	1D-Fixed M.U. Case 1		3D-Soil M.U. Case 1	
Modes	ω(Hz)	ω(Hz)	Δω(%)	ω(Hz)	Δω(%)
1st Transverse	0.81	0.86	-6.17	0.78	-3.85
1st Longitudinal	1.29	1.24	+3.88	1.13	-14.16
2nd Transverse	1.61	1.44	+10.56	1.50	-7.33
1st Bending	3.40	3.52	-3.53	3.47	+2.02
2nd Bending	3.46	3.56	-2.89	3.52	+1.70
3rd Bending	3.47	3.60	-3.75	3.65	+4.93
4th Bending	3.51	3.61	-2.85	3.69	+4.88
Average Δω(%)			+4.80		+5.55

Table 5: Modal frequencies identified via Ambient Vibrations (AV) and numerical modeling of the updated 1D-Fixed and 3D-3D Soil FEMs for structural parameter combination Case 1, and respective error (Δω%).

structural parameter	Model Updating Method Manual Case 2
E bearings	12.0
E deck	1.45
E piers	1.63

Table 6: Modal updating results for the 3D-3D Soil FEM through a proposed manual procedure.

The results of this parametric analysis resulted in the "Case 2" combination of updated structural parameters, summarized in Table 6. Table 7 shows the improvement of the modal frequencies predicted by the 3D-3D Soil for structural parameters combination "Case 2", compared to the modal frequencies predicted by the 1D-Fixed for "Case 1". By comparing Cases 1 and 2 (Tables 3 and 6), as well as the final modal frequencies predicted by the initial 1D-Fixed and the refined 3D-3D soil models, it is clear that the accurate soil-structure interaction modeling is a key parameter for reliable modal updating of the system.

It is also observed that both updated procedures (algorithmic and manual) showed that the Young Modulus of Elasticity for the bearings, the deck and the piers had to be significantly increased compared to the values assumed in the initial design. In particular, the bearing stiffness has to be increased by a factor of 12, while the concrete modulus of elasticity by a factor of 1.43 for the deck and 1.63 for the piers. With respect to the bearings, this can be clearly attributed to the low deformation (strain) levels that are developed under ambient vibrations at which the bearing stiffness is significantly lower than that assumed during design.

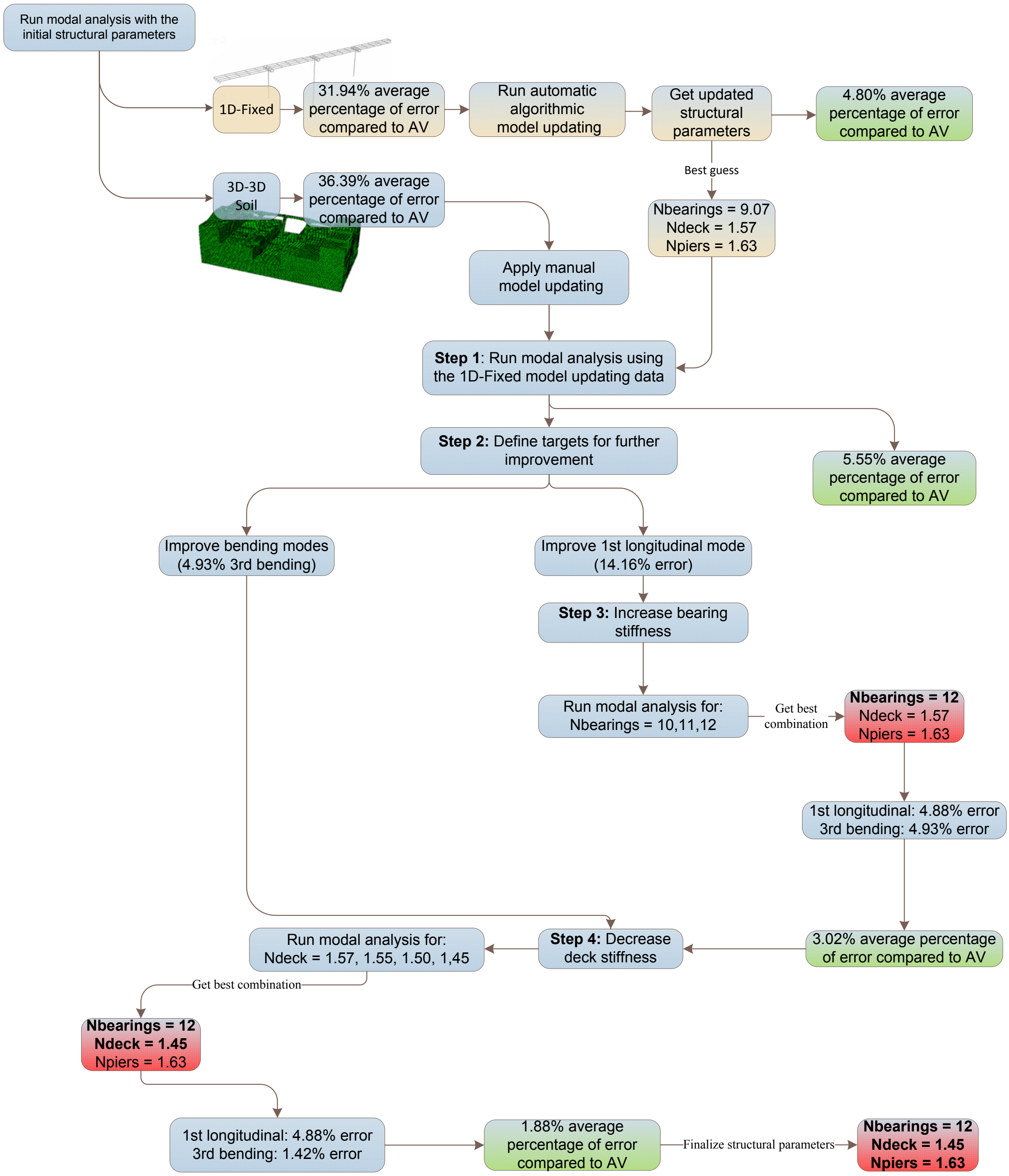

Figure 7: Manual model updating strategy.

The differences in the material properties of concrete can also be attributed to a series of contributing factors such as: (a) the definition of the modulus of elasticity according to the code used, which is calculated at strains higher than the ones imposed by ambient vibrations, (b) strengthening of concrete due to aging, (c) friction mechanisms that were activated at low levels of strain and (d) construction practices and quality control issues related to the casting of concrete. It is interesting to notice though, that similar deviations have been computed in numerous studies (i.e., [1]) and hence it is an issue that has to be investigated further.

	AV	1D-Fixed M.U. Case 1		3D-Soil M.U. Case 2	
Modes	ω(Hz)	ω(Hz)	Δω(%)	ω(Hz)	Δω(%)
1st Transverse	0.81	0.86	-6.17	0.79	-2.53
1st Longitudinal	1.29	1.24	+3.88	1.23	-4.88
2nd Transverse	1.61	1.44	+10.56	1.59	-1.26
1st Bending	3.40	3.52	-3.53	3.43	+0.87
2nd Bending	3.46	3.56	-2.89	3.46	+0.00
3rd Bending	3.47	3.60	-3.75	3.52	+1.42
4th Bending	3.51	3.61	-2.85	3.59	+2.23
Average Δω(%)			+4.80		+1.88

Table 7: Modal frequencies identified via Ambient Vibrations (AV) and numerical modeling of the updated 1D-Fixed for structural parameter combination Case 1and 3D-3D Soil FEMs for Case 2, and respective error (Δω%).

7 CONCLUSIONS

This paper aimed to identify the parameters that affect the dynamic response of the instrumented, 2nd Kavala Bypass Ravine Bridge constructed along the Egnatia Motorway Greece. Using alternative finite element models of various levels of complexity and modeling refinement in terms of consideration the dynamic interaction of the overall superstructure-foundation-soil and deck-abutment-embankment system, the modal frequencies of the bridge are computer and compared with the ones identified using ambient vibrations. The main conclusions drawn from this study can be summarized as follows:

- A good agreement was observed, on the basis of the predicted modal frequencies of the bridge, between the 1D-Fixed and the 3D-Fixed models, thus indicating that the simplifying assumptions made for the first were reasonably accurate. However, the initial prediction of the two models leads to considerably lower (34%-58%) modal frequencies than the ones identified, hence yielding model updating inevitable.
- Introduction of the soil compliance through the more refined 3D-3D soil model which simulated the overall soil-structure system led to a further reduction of the modal frequencies, by 10% in the 1st longitudinal direction and by 25% on average in the transverse direction.
- The model updating strategy that was followed for the case of the most complex and comprehensive 3D-3D soil model, eventually led to good agreement of the predicted modal frequencies with the identified ones and the average error was reduced to 1.88% (without exceeding 4.9% at any mode). The updated FEMs revealed that the differences

initially computed, were due to the higher actual stiffness of the elastomeric bearings, the piers and the deck as compared to the values that were assumed during design.

- The differences between the design assumptions and the actual structural properties under ambient vibrations can attributed to the low deformation (strain) levels, the definition of the modulus of elasticity according to the code used, which is calculated at strains higher than the ones imposed by ambient vibrations, strengthening of concrete due to aging, friction mechanisms as well as to construction practices during concrete casting.
- Consideration and numerical modeling of soil-structure interaction, abutment-backfill-embankment and pier-foundation-soil geometry and properties, may not affected the dynamic characteristics drastically in terms of their absolute values, but due to significant modal coupling, had a considerable effect on the prediction of the final, modified structural parameters. This effect is anticipated to be further pronounced in case of softer soil profiles. As a result, the accurate soil-structure interaction modeling is deemed to be a key parameter for reliable modal updating of extended bridge-soil systems.

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