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# MODEL UPDATING OF A BRIDGE-FOUNDATION-SOIL SYSTEM BASED ON AMBIENT VIBRATION DATA

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**Abstract.** *In Structural Health Monitoring (SHM) applications, observed discrepancies between identified and numerically calculated dynamic characteristics of structures have to be carefully interpreted and evaluated. Structural damage is not always responsible for those discrepancies, which can also be attributed to the initial assumptions adopted for the numerical models. In the present work a model updating framework is applied for a structure-foundation-soil experimental test–case, where discrepancies were observed between identified and numerically computed dynamic characteristics of an undamaged bridge-foundation-soil system. The scope is to interpret the observed discrepancies, to validate alternative numerical approaches of simulating soil-structure-interaction and to investigate how the refinement of the numerical models influences the model updating results. To this end, a steel strucure comprising an equivelnt scaled system of an under-construction isostatic deck-pier-caisson part of Metsovo bridge, in Greece, was studied in the laboratory. The Stochatsic Subspace Identification (SSI) method was implemented in order to identify the dynamic characteristics (natural frequencies, modeshapes, damping ratios) of the structure. Numerical models with different level of modeling complexity were developed and were then updated until correlation was achieved between identified and numerically calculated natural frequencies. The results indicated that the soil's initially adopted shear modulus G was overestimated thereby requiring appropriate calibration. After model updating takes place, all considered numerical models provide natural frequency estimates that are in good agreement with those identified based on ambient vibrations, indicating that less refined numerical models can be implemented in SHM applications in an efficient manner, provided soil parameters are accurately accounted for.*

# 1 INTRODUCTION

The management and maintenance of significant civil engineering technical projects is based nowadays to a great extent on Structural Health Monitoring applications (SHM). SHM involves mainly the identification of the dynamic characteristics of a structure over time, in order to monitor any changes to its structural components that may indicate structural damage. Furthermore, SHM involves the development of numerical models that perform similarly to the actual structural response, so that they can be utilized for its reliable assessment. The validation of the assumptions adopted to the nominal FE models is essential, given that models are inevitably governed by simplified assumptions and idealizations, regarding the simulation of the geometry, the material properties and the boundary conditions, leading commonly to discrepancies between the measured and the numerically predicted dynamic characteristics of a structure.

In cases where such discrepancies are observed and especially when the structure is founded on flexible soil deposits, a meaningful enhancement lies in the improvement of the simulation of the structure-foundation-soil stiffness, since error is likely to be introduced in the analysis when the soil-structure interaction effect is not accurately accounted for (Crouse et al. [1] and Chaudhary [2]). Advanced methods exist that consider the entire structure-foundation-soil system as a whole (Wolf [3]), but have high computational demands. Alternative methods have also been developed, involving kinematic and inertial decoupling of the superstructure-foundation-soil system by replacing soil with Winkler type springs. For the case of embedded foundations, a wide range of formulas have been proposed in the literature for the determination of the spring coefficients based on the foundation shapes and the foundation subsoil (Elsabee and Morray [4], Dominguez and Roesset [5], Gazetas et al [6], Gazetas and Gerolymos [7], Varun et al [8]).

The accurate simulation of the structure-foundation-stiffness is however linked both to the method used to represent the soil stiffness as well as to the determination of the actual soil properties. In this respect, one other common practice lies in the updating of the nominal soil parameters assigned to the FE models. Model updating uses the identified modal data and formulates an optimization problem in which the optimal values of the parameters of a FE model are commonly attained via minimization of a measure of the residuals between the measured and numerically predicted modal characteristics. Some of the available algorithms can be found in the work of Teughels et al. [9], Lam et al. [10], Christodoulou and Papadimitriou [11]).

Along these lines, in the present work a model updating framework is applied for a structure-foundation-soil experimental test–case, where discrepancies were observed between identified and numerically computed dynamic characteristics of an undamaged bridge-foundation-soil system [12]. Despite the fact that the soil-foundation stiffness was modeled using three distinct methods, of different level of modeling refinement, the observed discrepancies where of the same order (12%), indicating that the nominal adopted G shear modulus was overestimated. The scope of the present work is to interpret the observed discrepancies, to validate alternative numerical approaches of simulating soil-structure-interaction and to investigate how the refinement of the numerical models influence the model updating results.

# 2 BENCHMARK STRUCTURE

## 2.1 Structural system

The Metsovo ravine bridge is an already constructed reinforced concrete bridge located at the northwestern part of Egnatia Highway, in Greece. The bridge was constructed by the balanced cantilever construction method, which made feasible the modal identification of individual, structurally independent bridge segments during construction. Therefore, the modal characteristics of the $M_3$ pier-deck segment were identified by Panetsos et al. [13] prior to the construction of the key sections connecting it to the remaining segments of the bridge (Figure 1, background). As a result, the $M_3$ pier-deck segment was temporary acting as an independent balanced cantilever. The total length of the deck supported on the $M_3$ pier was, at the time that the measurements were obtained, 215m, while the height of the pier was 32m. The latter is founded via a large caisson embedded in thick interchanges of sandstone and limestone, which roughly correspond to soil class A according to Eurococe 8.

Utilizing the already identified modal characteristics of the M3 pier-deck segment, a scaled replica was constructed [12]. The scale was set to 1:100 mainly due to the long 215m deck of the T-shaped prototype cantilever and the laboratory space limitations. At this scale the construction of an exact replica of the concrete deck section was not feasible as it would result to web and flange dimensions as thin as 22 mm and 3 mm. Therefore, an equivalent scaled steel structure was designed to have similar dynamic characteristics with those of the prototype concrete $M_3$ pier-deck cantilever (Figure 2, only the superstructure). After appropriate dimensional optimization (using the FE software ABAQUS 6.12) relying on standard sections available in the market, the steel balanced cantilever was formed using the following commercially available sections: (a) a 90X90X3 HSS hollow steel section of 215cm length corresponding to 1:100 replication of the prototype deck, (b) a 100X100X5 HSS hollow steel section of 6,15cm length corresponding to the prototype central deck-segment, (c) two 80X20X3 HSS hollow steel sections of 32cm length corresponding to 1:100 replication of the prototype $M_3$ pier, and (d) a 100X100X5 steel plate that was used as the base of the pier.

In [12] the modal equivalence of the concrete M3 pier-deck segment with its steel replica is examined. The steel replica (Figure 2, only the superstructure) was fixed at its base to represent the stiff soil conditions of the prototype and its modal characteristics were identified based on low intense hummer impacts. It has been observed that the natural frequencies of the equivalent scaled structure present a 6.34% average deviation compared to those expected from the prototype's ideal 1:100 scaled structure, indicating good agreement between the equivalent (steel) and the prototype (concrete) bridge pier.

## 2.2 Soil conditions

Having established a level of confidence between the prototype structure and its equivalent replica, alternative soil conditions were applied to the scaled superstructure. In the present study the superstructure, as shown in Figure 2, was fixed on a circular concrete (C30/37) foundation (15cm diameter and height) that was embedded in stabilized soil. The stabilized soil consisted of clay (CL), 24% water and 4% lime. The total height of the soil deposit was 30cm and its dry density was determined as $\rho_S$=1.86t/m$^3$. Two sensors were placed inside soil, measuring the velocity of the shear waves ($V_S$) and two measuring the compression waves ($V_P$). In [12] it was shown that the $V_S$ measurements were not reliably measured leading to an overestimated value of $G_{nominal}$ (186MPa). In the present work the $V_P$ measurements were utilized in order to re-estimate the $G_{nominal}$. Finally, based on Equations 1 and 2 and by assuming a representative value v=0.35 for this type of soil, the $G_{nominal}$ was determined as 51MPa.

$$V_S = \sqrt{\frac{V_P^2 - 2vV_P^2}{2(1-v)}} \tag{1}$$

$$G = V_S{}^2 \rho \tag{2}$$


Figure1: Metsovo bridge $M_3$ pier-deck segment prior (background) and after its connection with the $M_2$ pier-deck segment (foreground).

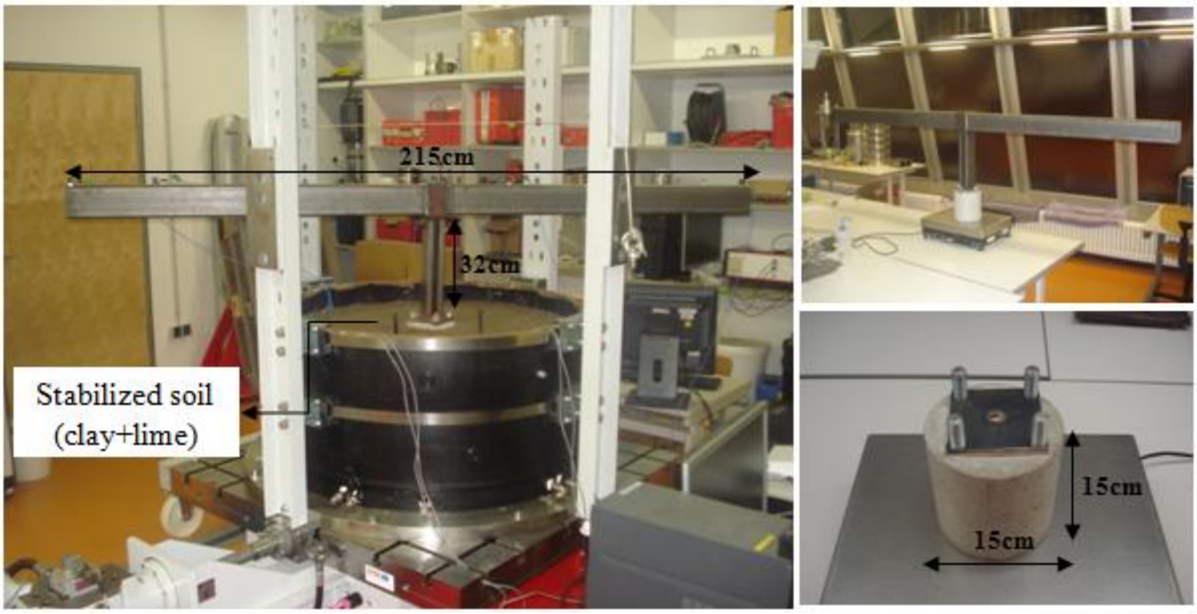



Figure 2: Steel scaled structure of the $M_3$ pier-deck segment founded on stabilized soil (left), detail of the laboratory model before its placement in the soil deposit (right, up) and detail of the concrete foundation (right, down).

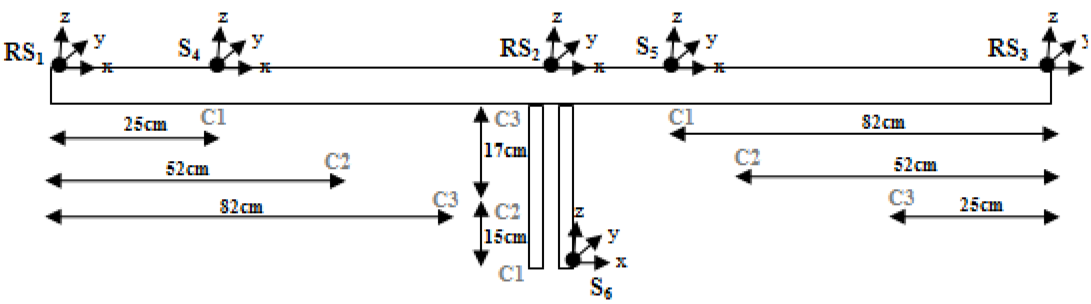



Figure 3: Arrangement of the 3 reference sensors $RS_1$-$RS_3$ and alternative positions of the portable sensors $S_4$-$S_6$ in Configurations C1, C2 and C3.

# 3 SYSTEM IDENTIFICATION

## 3.1 Instrumentation

The dynamic characteristics (natural frequencies, modeshapes, damping ratios) of the scaled structure founded on stabilized soil (Figure 2) were identified based on hammer impulses, to simulate a broadband excitation similar to ambient excitations applied to the actual $M_3$ pier-deck segment. The measurements were performed at the laboratory of Soil Mechanics at the Bauhaus University Weimar, in Germany. The soil was placed in a 95cm laboratory box that was fixed on a $1X1m^2$ shaking table. Six triaxial accelerometers (along the longitudinal, transverse and vertical directions) were installed on the structure: five on the deck and one at the base of the pier. Three alternative configurations were formed. Three of the six sensors, namely $RS_1$, $RS_2$ and $RS_3$ were considered to be the reference sensors (RS) having the same location (left side, middle and right side of deck respectively) at all configurations. The portable sensors $S_4$, $S_5$ and $S_6$ were placed in alternative positions. The locations of the portable sensors were (Figure 3):

- Configuration1 (C1): $S_4$ at 25cm, $S_5$ at 82cm and $S_6$ at base of pier,
- Configuration 2 (C2): $S_4$ at 52cm, $S_5$ at 52cm and $S_6$ at 15cm and
- Configuration 3 (C3): $S_4$ at 80cm, $S_5$ at 25cm and $S_6$ at 32cm.

## 3.2 Stochastic subspace identification

An output-only ambient vibration-based system identification was applied to the developed scaled structure. The natural frequencies were identified by means of the covariance-driven Stochastic Subspace Identification method (Peeters and De Roeck [14]) using MACEC (Reynders and De Roeck [15]), which is a Matlab toolbox for operational modal identification. Herein a brief overview of the method is provided in place of an in depth presentation, which lies beyond the scope of this paper.

To begin with, the dynamic behaviour of a discrete mechanical system consisting of n masses connected with springs and dampers is described by the following differential equation:

$$M\ddot{U}(t) + C\dot{U}(t) + KU(t) = F(t) \tag{3}$$

where M, C, K are the mass, damping and stiffness matrices and F(t) is the excitation force. In civil engineering structures, where parameters are distributed, Equation 3 is obtained as the finite element approximation of the system with only n degrees of freedom. Equation 3 need be manipulated for use with system identification methods since: (a) this equation is in continuous time whereas measurements are sampled at discrete-time instants, (b) it is impossible to measure all dofs as implied by this equation and (c) there is some modeling and measurement noise, which need be accounted for. In this respect, the Stochastic Subspace Identification method assumes that the dynamic behavior of a structure, excited by white noise, can be described at the $k^{th}$ sample time by the following discrete-time, linear, time-invariant, stochastic, state-space model of Equation 4:

$$x_{k+1} = Ax_k + w_k \tag{4a}$$

$$y_k = Cx_k + \upsilon_k \tag{4b}$$

where $x_k$ is the internal state vector, $y_k$ is the measurement vector, $w_k$ is the process noise due to disturbances and modeling inaccuracies, $\upsilon_k$ is the measurement noise due to sensor inaccuracy, A is the discrete state matrix containing the dynamics of the system and, C is the output matrix that translates the internal state of the system into observations. The state space matrices are then identified based on the measurements and by using QR-factorization, Singular Value Decomposition (SVD) and least squares robust numerical techniques. Once the state space model is found, the modal parameters (natural frequencies, damping ratios and modeshapes) are determined by an eigenvalue decomposition.

Five natural frequencies (Figure 5) were identified at the steel scaled structure founded on stabilized soil, namely a rotational one, the 1$^{st}$ longitudinal, a transverse one, the 2$^{nd}$ longitudinal and finally one bending mode.

## 4 NUMERICAL MODELS

### 4.1 Modelling of superstructure

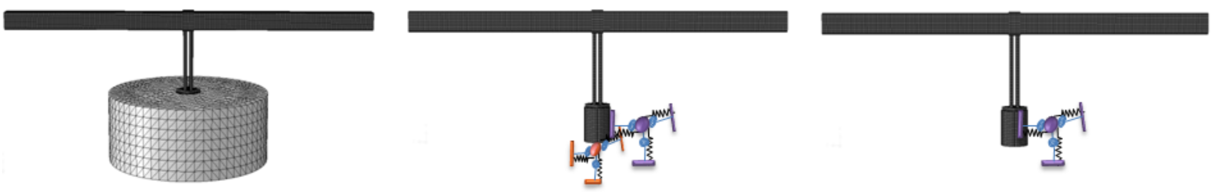
All numerical simulations were carried out with the FE-Code ABAQUS 6.12 [16]. Three FE models were developed. The resulting FE models simulated the superstructure of the constructed scaled model commonly using three-dimensional 8-node linear brick elements. The mesh size of the superstructure was 1X1 to all FE models. Stainless steel was assigned as the material of the superstructure with E=210GPa, v=0.3 and $\rho$=7.46t/m$^3$ (measured mass of superstructure 20.46kg).

### 4.2 Modelling of soil-foundation stiffness

The developed numerical models simulated the dynamic stiffness of the soil-foundation system with three alternative methods having different level of modeling complexity.

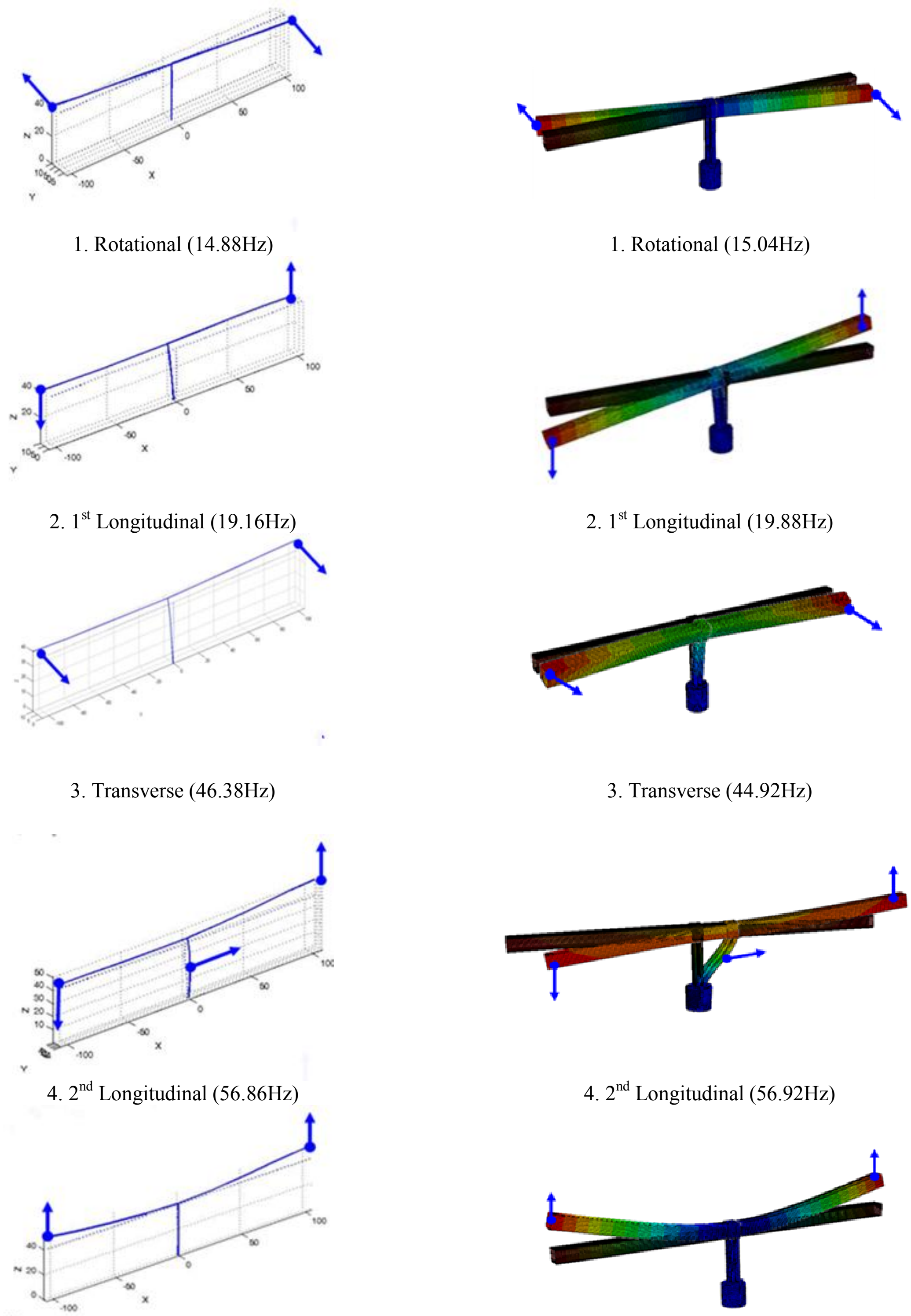
<u>4.2.1 Method 1: Holistic method</u>

In the first method the dynamic stiffness of the soil-foundation system was simulated as whole [3] in the three dimensional space using 8-node linear brick elements for the foundation and 10-node quadratic tetrahedron elements for the soil deposit (Figure 4.a). The mesh size of the foundation was 1X1 whereas the mesh size of the soil 5X5. In this approach, the resulted FE model had approximately 200,000 DOFs. Regarding the soil properties an isotropic homogenous material was assigned to the 3D soil volume with the properties that are mentioned in Section 2.2. A C30/37 class concrete (E=32GPa, v=0.30) was assigned to the caisson with density $\rho$=2.71t/m$^3$ (the mass of the foundation was measured 7.56kg).



(a) (b) (c)

Figure 4: Developed numerical models with alternative level of modeling complexity regarding the simulation of the soil-foundation stiffness: (a) a holistic method with 3D solid finite elements [3] (Method 1), (b) a 6+6 DOF springs method suggested by Kausel [17], and Varun et al [8] (Method 2) and (c) a 6-DOF spring method introduced by Elsabee et al. [4] (Method 3).



1. Rotational (14.88Hz) | 1. Rotational (15.04Hz)

2. 1st Longitudinal (19.16Hz) | 2. 1st Longitudinal (19.88Hz)

3. Transverse (46.38Hz) | 3. Transverse (44.92Hz)

4. 2nd Longitudinal (56.86Hz) | 4. 2nd Longitudinal (56.92Hz)

5. Bending (85.04Hz) | 5. Bending (87.15Hz)

Figure 5: Identified and numerically predicted modeshapes of the calibrated 6+6 DOF springs model (Method 2).

4.2.2 Method 2: Intermediate embedded circular foundation model

A second method to model the foundation-soil system was selected, based on the formulas proposed for intermediate embedded circular foundations (length-to-diameter aspect ratio 2<D/B<6). In this case, the foundation was simulated in the same manner as in the first method but the soil was now simulated via use of springs instead of a 3D soil volume (Figure 4.b). In particular, the subsoil at the tip of the caisson was modeled as a 6-DOF spring, while the stiffness of the surrounding soil was modeled by an additional 6-DOF spring assigned at the middle of the foundation height. The former 6-DOF stiffness matrices were obtained from the theory of surface circular foundations on a stratum over rigid base as suggested by Kausel [17] and the latter 6-DOF stiffness matrices by the solution of Varun et al. [8] for cylindrically shaped intermediate embedded foundations. In those formulas the 12 nominal spring coefficients were calculated based on the $G_{nominal}$ shear modulus that was determined by the $V_P$ sensors (51MPa).

4.2.3 Method 3: Shallow embedded cylindrical foundation model

In the third method selected herein, the foundation was once again simulated in the same manner as in the first case, whereas now the soil was replaced by 6-DOF Winkler type springs (Figure 4.c) in the middle of the caisson's height. The spring coefficients were obtained from the theory of shallow embedded cylindrical foundations (length-to-diameter aspect ratio D/B<2) placed on a homogenous soil stratum over bedrock, as proposed by Elsabee and Morray [4]. Again the nominal measured shear modulus $G_{nominal}$=51MPa was utilized to determine the 6 spring coefficients.

## 5 CALIBRATION OF NUMERICAL MODELS

### 5.1 Method

A FE model updating application aims to estimate the values of the structural parameter $\theta$ of a developed numerical model so that the natural frequencies and mode shapes {$\omega_r(\theta)$, $\phi_r(\theta)$, *r=1,..,m*} predicted by this model accurately approximate the experimentally obtained modal characteristics {$\hat{\omega}_r$, $\hat{\phi}_r$, *r=1,..,m*}, where m is the number of modes of interest. The objective function J(θ) represents an overall measure of fit between the measured and the model predicted modal characteristics. Namely, the first norm in Equation 5 represents the measure of fit 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 eigenvector for the r -th mode, through the Modal Assurance Criterion (MAC). In Equation 6 the closest the MAC value is to one the closer the fit and the minimum the value of the second norm in Equation 5. Herein, no weighted values were implemented to the two norms since the assumption of equal weighted residuals is adopted.

$$J(\theta) = \sum_{r=1}^{m} \left[ \frac{\left[\omega_r(\theta) - \hat{\omega}_r\right]^2}{\left[\hat{\omega}_r\right]^2} \right] + \sum_{r=1}^{m} \left[1 - \left|MAC_r(\theta)\right|\right] \tag{5}$$

$$\left|MAC_r(\theta)\right| = \left| \frac{\phi_r(\theta)^T \times \hat{\phi}_r}{\left\|\phi_r(\theta)\right\| \times \left\|\hat{\phi}_r\right\|} \right| \tag{6}$$

The matlab optimization toolbox [18] was utilized for the minimization of the objective function J(θ). The *fminserach* algorithm was selected to be the optimization algorithm. This algorithm is an unconstrained nonlinear minimization algorithm that uses the Nelder-Mead simplex algorithm as described in Lagarias et al. [19].

### 5.2 Calibrated soil-foundation stiffness

In [12] the natural frequencies of the steel replica of the $M_3$ pier-deck segment were identified under fixed support conditions. The respective developed numerical model predicted natural frequencies corresponding to an average error as lows as 2.12%. This verified that the nominal values assigned to the material properties of the superstructure were realistic and that they do not need to be subjected to calibration. Moreover, since there was no uncertainty concerning the modelling of the superstructure, the discrepancies observed between the measured and model-predicted natural frequencies for the case of the stabilized soil were attributed to the determination of the actual soil stiffness. Therefore, the parameter θ selected to be updated is the soil's stiffness related parameter that scales the contribution of the G nominal value that was assigned to the initial FE models. Thus, the nominal FE models correspond to parameter values θ=1.

The model updating results for the steel scaled structure of the $M_3$ pier-deck segment of Metsovo bridge that was founded on stabilized soil are presented in Table 1. As shown in Table 1 the average percentage error Δω between the measured modal frequencies and the modal frequencies predicted by the three optimal FE models is 2.10% for the 3D-Soil FE model (Method 1), 2.10% for the 6+6 DOF springs FE model (Method 2) and 2.33% for the 6-DOF springs FE model (Method 3). The measured average error Δω is of the same order for all FE models with the 2.12% average error Δω measured at the fixed case, where no assumptions regarding the soil conditions were adopted, verifying the successful model updating. Furthermore the Modal Assurance Criterion (MAC) was utilized in order to compare the measured mode shapes with those numerically predicted. As observed in Table 1 all MAC values are close to 1 indicating the good agreement between the measured and the model predicted mode shapes for all three optimal FE models.

| | | Steel scaled structure founded on stabilized soil | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Identified | | FE models (calibrated) | | | | | |
| | | SSI | | Method 1: 3D-Soil θ=0.83 | | Method 2: 6+6 DOF springs θ=0.81 | | Method 3: 6-DOF springs θ=0.87 | |
| No | Modeshape | f(Hz) | ζ(%) | f(Hz) | MAC | f(Hz) | MAC | f(Hz) | MAC |
| 1 | Rotational | 14.88 | 0.37 | 15.05 | 0.99 | 15.04 | 0.99 | 15.09 | 0.99 |
| 2 | $1^{st}$ Longitudinal | 19.16 | 0.85 | 19.74 | 0.99 | 19.88 | 0.99 | 19.54 | 0.99 |
| 3 | Transverse | 46.38 | 1.56 | 44.84 | 0.99 | 44.92 | 0.99 | 44.61 | 0.99 |
| 4 | $2^{nd}$ Longitudinal | 56.86 | 3.27 | 57.34 | 0.93 | 56.92 | 0.93 | 57.77 | 0.93 |
| 5 | Bending | 85.04 | 1.81 | 86.88 | 0.99 | 87.15 | 0.99 | 87.33 | 0.99 |
| | Average Δf(%) | | | 2.10 | | 2.10 | | 2.33 | |

*where θ is the $G_{optimal}$ to the $G_{nominal\ aspect\ ratio}$*

Table 1: Natural frequencies of the steel scaled structure founded on stabilized soil, identified with the Stochastic Subspace Identification method (SSI) versus the calibrated numerical predictions of the three developed numerical models.

The above-mentioned results were obtained after the calibration of the soil's G shear modulus. Table 1 presents the θ values that were predicted by the updating algorithm for all developed numerical models. Since θ is actually the $G_{optimal}$ to $G_{nominal}$ aspect ratio and the $G_{nominal}$ was measured in laboratory 51MPa, the values θ=0.83, θ=0.81 and θ=0.87 correspond to $G_{optimal}$=43MPa for the 3D-Soil model of Method 1, $G_{optimal}$=41MPa for the 6+6 DOF springs model of Method 2 and $G_{optimal}$=45MPa for the 6-DOF springs model of Method 3, respectively. The optimal values of the soil's G shear modulus (41-45MPa) are in good agreement at all numerical models despite their different level of modeling refinement, demonstrating their equivalence in representing soil-foundation stiffness. Furthermore, all calibrated numerical models predicted that the soil's nominal G shear modulus had to be reduced by 19-13%. This demonstrates that the $V_P$ were more reliably measured than the $V_S$ since now the 51 MPa nominal G shear value is much closer to the optimal ones (41-45MPa), compared to the $G_{nominal}$=186MPa calculated based on the $V_S$ measurements. However, the inherent uncertainty associated with the measurement of soil properties, even under laboratory control conditions is highlighted, in both measurements.

Based on the above-mentioned results it is evident that all developed FE models managed to successfully represent the soil-foundation stiffness after the optimal calibration of the measured G shear modulus. Eventually, it may be suggested that after proper updating, Winkler type models (Method 2 and Method 3) are capable of accurately accounting soil-foundation stiffness at low computational cost, compared to the more refined 3D holistic models (Method 1) at least for cohesive soil deposits as clay is.

## 6 CONCLUSIONS

In the present work a model updating framework is applied for a structure-foundation-soil experimental test–case, where discrepancies were observed between identified and numerically computed dynamic characteristics of an undamaged bridge-foundation-soil system. The scope lies in interpreting the observed discrepancies, validating alternative numerical approaches of simulating soil-structure-interaction and in investigating how the refinement of the numerical models influences the model updating results. In this context, the dynamic characteristics (natural frequencies, modeshapes, damping ratios) of a steel scaled replica of the under construction $M_3$ pier-deck segment of Metsovo bridge were identified in laboratory based on low intense hummer impacts. The structure was studied for two cases of soil conditions (stabilized soil and Hostun sand). Three methods were adopted to simulate the soil compliance of the stabilized soil, namely: (a) a holistic method with 3D solid finite elements [3], (b) a 6+6 DOF springs method suggested by Kausel [17] and Varun et al [8] and (c) a 6-DOF spring method introduced by Elsabee et al. [9]. The conclusions drawn after the model updating can be summarized as follows:

- The updated numerical models predict natural frequencies and mode shapes that are in good agreement with those identified by the ambient vibration measurements, having 2% average error, after the calibration of the soil's G shear modulus.

- The optimal value of soil's G shear modulus was reliably identified given that numerical models of different refinement predicted after calibration similar values ($G_{optimal}$=41-45MPa). The results showed that soil's nominal G shear modulus was overestimated and that it had to be approximately 20% reduced. The observed discrepancies between $G_{optimal}$ and $G_{nominal}$ values highlight the inherent uncertainty associated with the measurement of soil properties even under laboratory control conditions (heterogeneous soil, sensor sensitivity, process error).

- The fact that the updated results rendered similar $G_{optimal}$ values (41-45MPa) for all developed numerical models indicates additionally that the refinement of the soil-stiffness did not influence radically the model updating results, as expected. This verifies the efficiency of the suggested methods in simulating soil-foundation stiffness and their modeling equivalence.
- Eventually, it may be suggested that at least for the case of cohesive soil types such as clay, simpler Winkler-type models (Method 2 and Method 3), are adequately capable of numerically predicting soil stiffness at low computational cost compared to the 3D holistic methods (Method 1), when reasonable assumptions are adopted regarding the soil properties.

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