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Title: *Structural health monitoring of a ravine bridge of Egnatia Motorway during construction.*

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ABSTRACT

The instrumental rapid monitoring of the dynamic (ambient) response of a balanced cantilevered ravine bridge of Egnatia Motorway during its construction phases, when subjected to wind and other construction loads, was implemented. The aim is to verify the conformity both of the sequential construction phases and of the final completed structure of the ravine bridge of Metsovo to the design predictions. In this paper the modal frequencies, damping ratios and modeshape components of the completed balanced cantilever of pier M3 were identified from ambient acceleration records, and its analytical dynamic model was updated to determine the actual stiffness and mass properties of the structure.

INTRODUCTION

In Northern Greece the largest and most challenging Greek project of design, supervision, construction, operation, maintenance and exploitation of 680 km of the motorway, linking Europe with Turkish borders, has been almost constructed. This is the Egnatia Motorway (E.M.) project.

EGNATIA ODOS S.A. (E.O), the company responsible for the design, construction, maintenance and exploitation of E.M. developed an integrated Bridge Management System for optimizing the maintenance and repair policies for bridges of the motorway. In the last years, the initial visual inspection of all the newly established bridges was done, in combination with the instrumental monitoring of some of the major bridges of the motorway, to give initial structural and functional

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condition data.

Some major bridges over steep and deep ravines, in the west sector of the E.M., crossing particularly difficult geological terrain and obstacles, will be the last to be constructed, for completing the project. In this paper the first results from the rapid instrumental monitoring of the ambient vibration of one of these major ravine bridges of the west sector of E.M., during its construction phases are presented.

INSTRUMENTAL MONITORING OF METSOVO RAVINE BRIDGE

The new under construction ravine bridge of Metsovo (Figure 1), in section 3.2 (Anthohori tunnel-Anilio tunnel) of E.M., is crossing the deep ravine of Metsovitikos river, 150m over the riverbed. This is the higher bridge of E.M., with the height of the taller pier M2 equal to 110m. The total length of the bridge is 357m. As a consequence of the strong inequality of the heights of the two basic piers of the bridge, M2 and M3 (110m to 35m), the very long central span of 235m, is even longer during construction, as the pier M2 balanced cantlilever is 250m long, due to the eccentrical position of the key segment. The key of the central span is not in midspan due to the different heights of the superstructure at its supports to the adjacent piers (13,0m in pier M2 and 11,50 in pier M3) for redistributing mass and load in favor of the short pier M3 and thus relaxing strong structural abnormality. The last was the main reason of this bridge to be designed to resist earthquakes fully elastic (q factor equal to 1).

The bridge has 4 spans, of length 44,78m /117,87m /235,00m/140,00m and three piers of which M1, 45m high, supports the boxbeam superstructure through pot
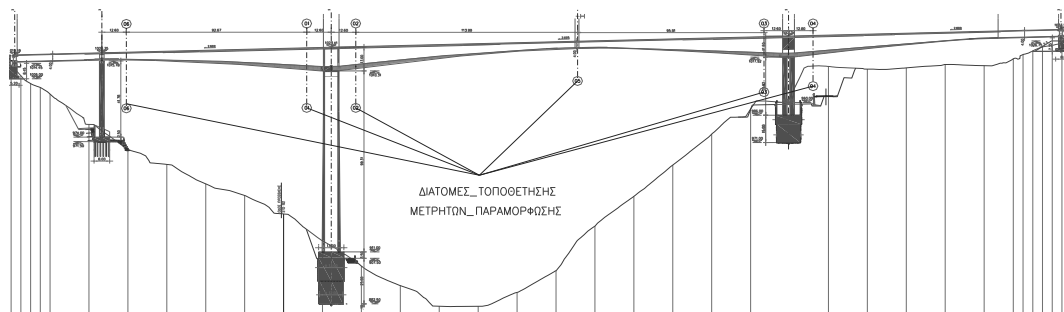

Figure 1. Longitudinal section of Metsovo ravine bridge of Egnatia Motorway.

Figure 2. Views of under construction Metsovo ravine bridge a) General view b) key of central span.

bearings (movable in both horizontal directions), while M2, M3 piers connect monolithically to the superstructure. The bridge is being constructed by the balanced cantilever method of construction and according to the constructional phases shown in Figure 2. The total width of the deck is 13,95m, for each carriageway. The superstructure is limited prestressed of single boxbeam section, of height varying from the maximum 13,5m in its support to pier M2 to the minimum 4,00m in key section.

The pier M3 balanced cantilever has been instrumented after the construction of all its segments and before the construction of the key segment that will join with the balanced cantilever of pier M2 (Figure 3). The total length of M3 cantilever was at the time of its instrumentation 215m while its total height is 35m. Piers M2, M3 are founded on huge circular Ø12,0m rock sockets in the steep slopes of the ravine of the Metsovitikos river, in a depth of 25m and 15m, respectively.

Six uniaxial accelerometers were installed inside the box beam cantilever M3 of the left carriageway of Metsovo ravine bridge. The accelerometer arrays, are shown in Figure 3. Due to the symmetry of the construction method (balanced cavtilevering) and as the same number of segments were completed on both sides of pier M3, the instrumentation was limited to the right cantilever of pier M3, following two basic arrangements: a) according to the 1st arrangement two (2) sensors were supported on the head of pier M3, one measuring longitudinal and the other transverse accelerations (M3L, M3T), while the remaining four (4) accelerometers were supported on the right and the left internal sides of the box beam's webs , two (2) distant 46m and two (2) distant 68m from M3 axis, respectively (LV3, RV4, LV5, RV6). All four were measuring vertical acceleration. b) according to the 2nd arrangement the last two sensors of the 1st arrangement were fixed in a section near the cantilever edge, distant 93m from M3 axis, while the other four remain in the same positions. In both arrangements the sixth sensor was adjusted to alternatively measure both in vertical and in transverse horizontal direction (RV6 or RT6).

IDENTIFICATION OF THE MODAL CHARACTERISTICS OF THE METSOVO BRIDGE

The response of the cantilever structure subjected to ambient loads as the wind, and loads induced by construction activities as the crossing of light vehicles placing the prestressing cables inside the tendon tubes, was as expected of very low intensity (0,6% of the acceleration of gravity). The acceleration response time histories,
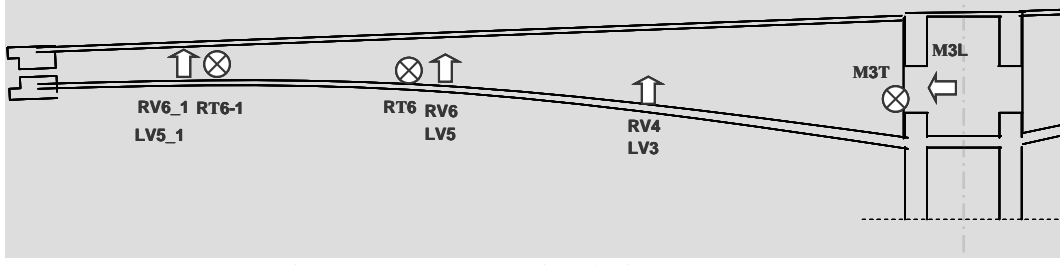
Figure 3. Accelerometer installation arrangements.

measured from the 6 channel arrays, were analyzed using the user friendly modal identification software, developed by the System Dynamics Laboratory of the University of Thessaly in cooperation with Egnatia Odos. All the basic modal frequencies, modeshapes and damping ratios of the bridge were identified.

In the so developed modal identification methodologies based on ambient vibrations, processing output only data, the excitation is considered as white noise stochastic process. The estimation of modal characteristics is achieved using the method of least squares. Specifically the identification is achieved by minimizing a weighted measure of fit

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

between the cross power spectral density functions (CPSD) $\hat{\mathbf{S}}(k\Delta\omega) \in C^{N_0 \times N_0}$ that are computed form measured response time histories and the CPSD functions $\mathbf{S}(k\Delta\omega;\boldsymbol{\psi}) \in C^{N_0 \times N_0}$ predicted by a modal model, where N_0 is the number of the measured degrees of freedom (DOF), $\Delta\omega$ is the step in the discretized frequency array, $k = l,...,N_\omega$ are the indices respecting to frequencies $\omega = k\Delta\omega$, N_ω is the number of discrete points in the frequency range, $\boldsymbol{w} \in R^{N_0 \times N_0}$ the matrix including all the weight coefficients and $\boldsymbol{\psi}$ is the vector of the parameters to be identified. For the solution of the minimization problem a three steps algorithm is being used, which is analytically described in [1].

The identified values of the modal frequencies and the corresponding values of the damping ratios are shown in Table I. Due to space limitations only ten of the identified modal frequencies and three of the modeshapes are presented in Table I and Figure 4. The arrows are placed in measuring points and their length is proportional to the respective value of the normalized modal component. The accuracy in the estimation of the modal characteristics is shown in Figure 5 comparing the measured with the modal model predicted CPSD.

TABLE I. IDENTIFIED MODES OF THE METSOVO RAVINE BRIDGE

No	Measured Identified frequencies	Measured Hz	Measured Damping ζ%	Design Model Model frequencies
1	1st rot, z axis	0.15	2.93	0.136
2	1st longitudinal	0.30	0.18	0.267

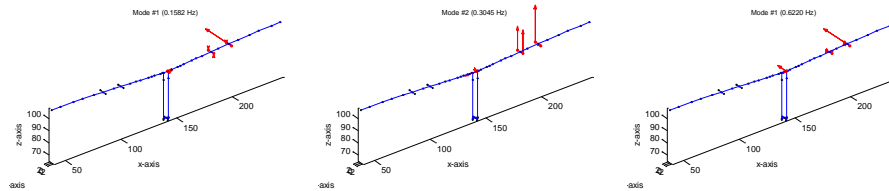| 3	1st transverse	0.62	0.43	0.511
4	2nd longitudinal	0.68	0.42	0.617
5	1st bending (deck)	0.90	0.25	0.70
6	2nd transverse	1.30	0.40	1.24
7	2nd bending (deck)	1.43	0.43	1.84
8	2nd rot,z axis	1.46	0.47	2.04
9	3rd bending (deck)	2.28	0.40	2.71
10	3rd transverse	2.58	1.27	3.79

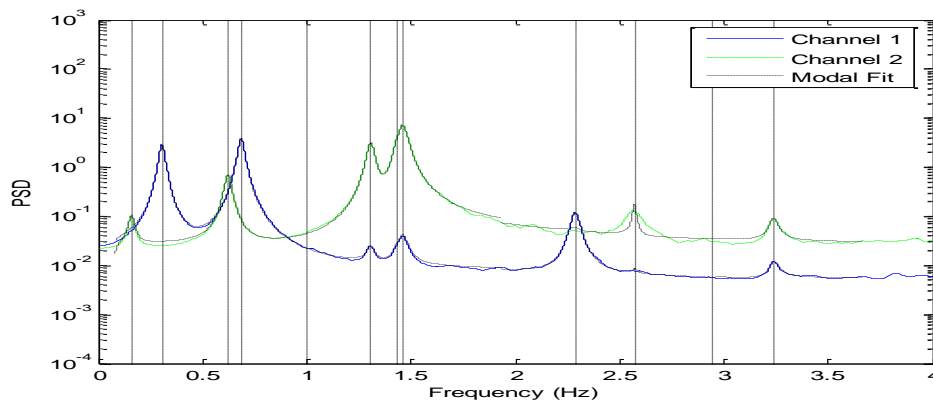Figure 4. The three first identified modeshapes of Metsovo bridge.

Figure 5.Comparison between measured and modal model predicted CPSD.

UPDATING OF THE DYNAMIC MODEL OF METSOVO BRIDGE FOR EXPERIMENTALLY DETERMINED MODAL CHARACTERISTICS

Three different analytical dynamic models of the bridge cantilever M3 were constructed, representing material and geometry of the structure, as considered by the design. For bridge modeling the software packages COMSOL Multiphysics, SAP2000NL and STATIK were used. Three dimension Euler beam finite elements were used for the construction of these models, coincided with the axis connecting the centoids of the deck and pier sections. For better graphical representation of the higher modeshapes, on the measured points of the bridge cantilever (positions of sensors), additional rigid transverse extensions of no mass were added to both sides of its centroid axis. For representing rigid connection of the superstructure to the pier, rigid elements of no mass were used. Analytical models shown in Figure 6 have 594, 264 and 448 degrees of freedom, respectively. Pier transverse webs were simulated by beam elements according to design drawings. For piers foundation, lateral and rotational springs at the basement of the piers were considered, such as to represent fixing conditions of piers to the huge circular rock sockets, in both

horizontal directions. The aim was to examine the contribution of soil conditions on the dynamic response of the bridge pier cantilever as well.
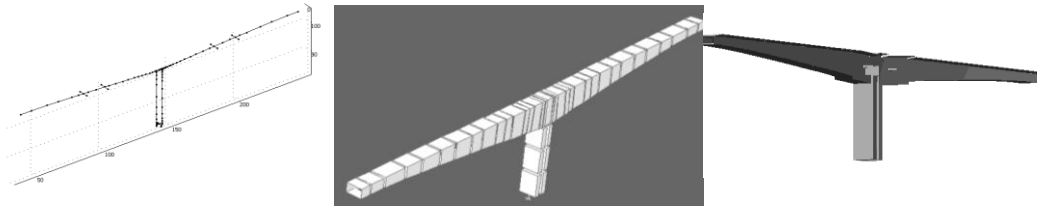

Figure 6. Dynamic models of cantilever M3 of Metsovo bridge with euler beam finite elements.
a) FEMLAB, b) SAP2000NL, c) STATIK

A methodology for updating the design dynamic model of the bridge, described before, was used, based on the experimentally identified modal data [2-3]. According to this methodology, the initial finite element model is parameterized by a parameter set which represent mass and stiffness properties at an element or substructure level. Such finite element properties to be parameterized could be the elasticity modulus multiplied by the moment of inertia for the superstructure and piers (E*I), spring constants simulating elastomeric bearings (GA/h), and others.

The objective in a modal-based model updating methodology is to estimate the values of the parameter set so that the modal properties generated by the finite element model best matches the experimentally obtained modal properties. The used method for model updating, searches for the optimal model parameters that minimize a measure of fit between the modal frequencies or/and modeshapes predicted by the finite element model at the measured degrees of freedom and the measured modal frequencies and modeshape components. Parameter estimation problems based on measured modal data are thus formulated as weighted least-squares problems in which objective functions

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

measuring the fit between measured $\hat{\omega}_r$, $\hat{\phi}_r$ and model predicted $\omega_r(\boldsymbol{\theta})$, $\phi_r(\boldsymbol{\theta})$ modal data, are build up into a single objective using weighting factors w [4]. Standard optimization techniques are then used to find the optimal values $\boldsymbol{\theta}$ of the parameters that minimize the overall measure of fit $J(\boldsymbol{\theta};\boldsymbol{w})$ [5]. Various weighted least-squares methods are integrated into the software.

The 1st modeshape (rotation of the deck round the piers M3) (0.1461 Hz) and the 5th modeshape (bending of the deck) (0.7257 Hz), computed by the analytical finite element models, are presented in Figure 7. The observation of the graphical representation of these modeshapes, leads to the conclusion that the stiffness of the piers governs the vibration of the bridge under the 1st mode, while the stiffness of the deck governs the vibration of th bridge under the 2nd mode.

For updating of the initial dynamic model of the bridge, the methodology described above, was first applied only for the experimental modal data of the first
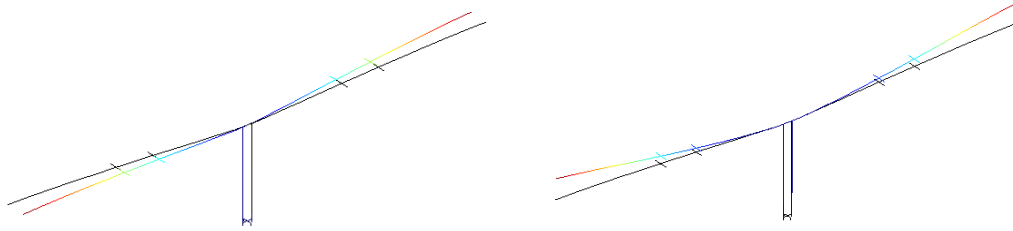

Figure 7: Modeshapes predicted by the finite element model of Metsovo bridge (a) 1st (rotational) (ω = 0.1461 Hz), (β) 5th (bending of the cantilevered deck ω = 0.7257 Hz).

and the fifth modal frequencies, targeting to update with accuracy the stiffness of piers and of the deck. The parameterized finite element model has two parameters θ_1 and θ_2. The first parameter θ_1 describes the stiffness of the piers, while the second parameter θ_2 describes the stiffness of the deck. These parameters multiply the values of the selected model properties that describe, such as the values $\theta_1 = \theta_2 = 1$ correspond to the initial model of the as designed bridge.

In Table II the optimal values $\hat{\theta}$ of the stiffness parameters of the piers and the deck, based on modal data of 1st and 5th modes, are given. According to the results, the actual stiffness of the piers is 1.2 times the initial stiffness of the design model, while the actual stiffness of the deck is 1.57 times the initial stiffness of the design model. The finite element model that corresponds to the optimal values of parameters θ_1 και θ_2 is considered now as the nominal model of the bridge. In Table II the values of the modal frequencies predicted by the nominal updated finite element model are given in comparison with the respective values of the experimentally identified modal frequencies, using data from the first and the fifth modes. The percentage errors ($\|\Delta\omega\| = (\omega - \hat{\omega})/\hat{\omega}$), between measured and model predicted frequency values, are here zero, which means that the updated model predicts the bridge vibration identical to the measured one.

Next, the updating of the nominal model of the bridge with three parameters is presented. The model updating is now carried out for the first 5 modal frequencies that have been experimentally identified. The nominal model is parameterized by three parameters θ_1, θ_2 and θ_3. The first parameter θ_1 describes the stiffness of the piers, the second parameter θ_2 describes the stiffness of the deck, while the third parameter θ_3 describes the stiffness of the soil springs of the pier foundation. These parameters multiply the values of the selected model properties that describe, such as the values $\theta_1 = \theta_2 = \theta_3 = 1$ correspond to the nominal model of the bridge.

TABLE II. OPTIMAL VALUES OF PARAMETERS θ , MEASURED VERS PREDICTED FREQUENCIES

Parameter	Value	Mode	Measured	Updated	$\Delta\omega$ %
E column	1.2032	1st	0.1592	0.1592	0%
E deck	1.5694	5th	0.9082	0.9082	0%

In Table III the optimal values $\hat{\theta}$ of the stiffness parameters of the piers, the deck and the soil springs, based on the modal data of the first 5 modes are given. According to the results of Table III, the actual stiffness of the piers and the deck are very close to the initial. It also means that for the excitation level of the bridge during monitoring the piers were performed as fixed to their foundations. In Table III the values of the modal frequencies predicted by the updated finite element model are given in comparison with the respective values of the experimentally identified frequencies, using data form the first five modes. The percentage errors ($\|\Delta\omega\| = (\omega - \hat{\omega})/\hat{\omega}$), between measured and model predicted frequency values, that are given as well in Table III, vary in the range of 0.2% to 1.8%, which means that the finally updated model is adequate to predict the bridge vibration according to the first five measured modes.

TABLE III OPTIMAL VALUES OF θ , MEASURED VERSUS PREDICTED FREQUENCIES

Parameter	Value	Mode	Measured	Nominal	Updated	$\Delta\omega$ %
E deck	0.9944	1st	0.1592	0.1592	0.1612	1.2563
E columns	0.9979	2nd	0.3049	0.3062	0.2992	1.8695
E spings	0.9700	3rd	0.6232	0.6670	0.6300	1.0911
		4th	0.6855	0.6738	0.6820	0.5106
		5th	0.9082	0.9082	0.9061	0.2312

CONCLUSIONS

In the present paper the fist results of the rapid ambient vibration monitoring of a ravine bridge of E.M., during construction, are presented. The successful identification of all the basic frequencies and the modeshapes of the cantilevered structure, enabled the updating of the model design values of the pier, the deck and the soil springs stiffness. The excellent fit to the measured modal frequencies leads to the conclusion that the proposed methodology can be very useful for verifying the as built condition of major bridges during their construction phases.

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