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CALIBRATION OF THE DYNAMIC MODEL OF A LONG CONCRETE RAVINE BRIDGE BASED ON AMBIENT VIBRATION MEASUREMENTS

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Abstract

The scope of this work is the creation of a numerical FE model of an existing prestressed post-tensioned RC box girder ductile ravine bridge and its updating, based on identified modal characteristics (eigenfrequencies). The challenge for this task rose from the fact that the identified modal characteristics were not adequate for the use of one of the advanced automated model updating methods that are available. In particular, the only usable parameters that were identified were some natural frequencies and their corresponding mode types but not eigenvectors. Safely assuming modern building conditions, the absence of localized structural damage and the linear behavior during the identification measurements, an empirical manual updating procedure is implemented for updating standard key parameters of the FE model. During the updating procedure the identified quantities are used to define the models uncertain parameters such as moduli of elasticity and boundary conditions, whilst robust engineering reasoning is implemented for the best and safest exploitation of the available identified modal data.

Keywords: Concrete Bridge; Updating; Monitoring system; Ambient vibration measurement

1. Introduction

In the case of various large scale civil engineering structures it is very difficult and often impossible to have an accurate estimation of the parameters of the finished product during its design phase. This results, more often than not, in serious discrepancies between the predicted behavior during design and the final behavior of the completed structure. As the demand for more sophisticated designs rise with the materials and building technologies, the need for minimizing these discrepancies increases. New identification procedures have been developed employable to large scale structures that determine the various dynamic characteristics, thus giving the necessary tools to the engineer for accurately identifying the actual state of the structure and additionally giving the necessary feedback to evaluate and evolve the various methodologies used for designing it.

The identification data combined with model updating techniques make possible the creation of mathematical models that can closely predict and evaluate the behavior of the structure under different loading scenarios, identifying and localizing potential structural damage and generally tracking the structures state in time.

The scope of this work is the creation of a numerical FE model of an existing prestressed post-tensioned RC box girder bridge and its updating based on identified modal characteristics (eigenfrequencies). The concrete ravine bridge is the G10 ravine bridge, bearing the left branch of the Egnatia Motorway, in Northern Greece. The bridge has three spans, built by the balanced cantilevering method.

Creep, shrinkage, thermal loads, fast curing concrete to follow the strict step by step balanced cantilevering construction schedule, jacking for adjusting the final cantilevers elevations (over the abutments) are some of the key factors that affect the mechanical properties of the completed bridge structure, set in operation 12 years ago. The identification of the actual properties of the major elements of the bridge is therefore very useful to evaluate and clarify the effect of the previously mentioned factors in the actual dynamic performance and the structural condition of the bridge.

The challenge for the particular task rose from the fact that the identified modal characteristics were not adequate for the use of one of the advanced automated model updating methods that are available. In particular, the only usable parameters that were identified were some natural frequencies and their corresponding mode types but not eigenvectors. Safely assuming modern building conditions, the absence of localized structural damage and the linear behavior during the identification measurements, an empirical manual updating procedure was implemented for updating standard key parameters of the FE model. During the updating procedure the identified quantities were used to define the models uncertain parameters such as moduli of elasticity and boundary conditions, whilst robust engineering reasoning was implemented for the best and safest exploitation of the available identified modal data.

2. The Structure's characteristics

The modeled structure is a prestressed RC box girder ravine bridge founded on weathered rock with three spans built via the balanced cantilever method around 2004. Bearing the right branch of the Egnatia motorway in northern Greece, the three spanned deck is supported monolithically on two hollow rectangular 45.1 and 41.3 meters high RC piers and via pot bearings on the two abutments. The piers are founded on deep cylindrical RC shafts embedded in weathered rock. The decks longitudinal axis is a circular arc of radius R=1506.25m and has total length of 234.2m. The three spans have lengths of 61.05m, 112.1m and 61.05m (figs. 1&2).

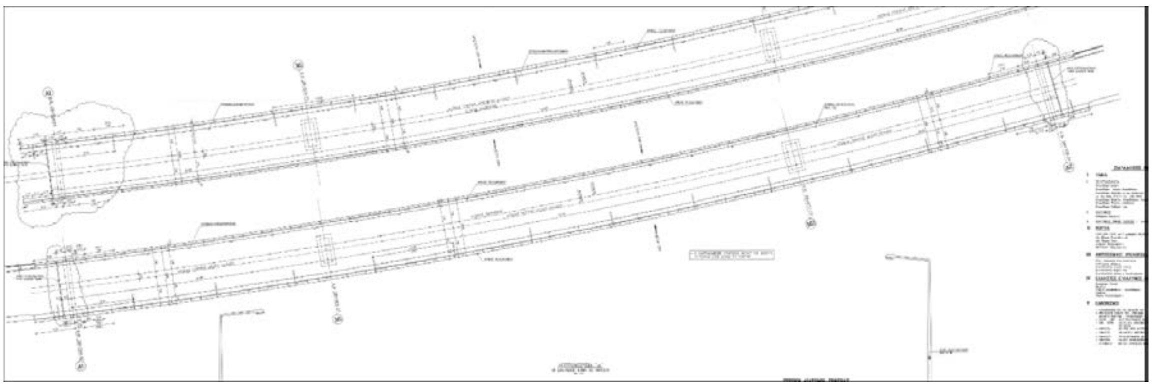

Fig. 1 –Plan view of both branches of G10 Egnatia motorway bridge.

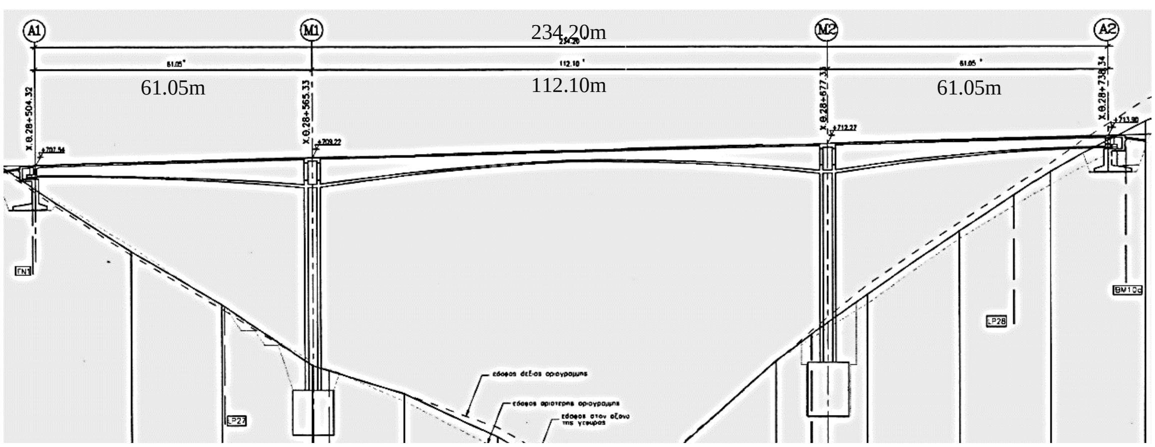

Fig. 2 – Vertical section of the bridge, on deck axis.

The deck has a constant longitudinal slope of i_L=2.7% and inward transverse slope that varies from 3.9% near the abutments to 5% at the central span. The transverse slope is followed by the top and bottom walls of the deck's box girder section (fig. 3). The height of the box girder section varies parabolically with maximum height at the piers and minimum in the middle of the middle span and at the abutments (figs. 2&4). The piers and pots are placed in plan in a polar manner with the piers weak axis being parallel to the decks radius and their strong axis parallel to the tangent of the decks arc.

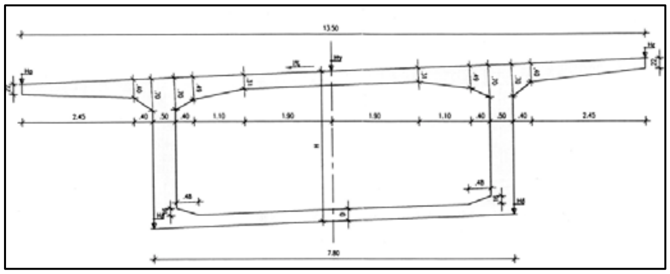

Fig. 3 – Typical box girder section illustrating the deck's transverse slope.

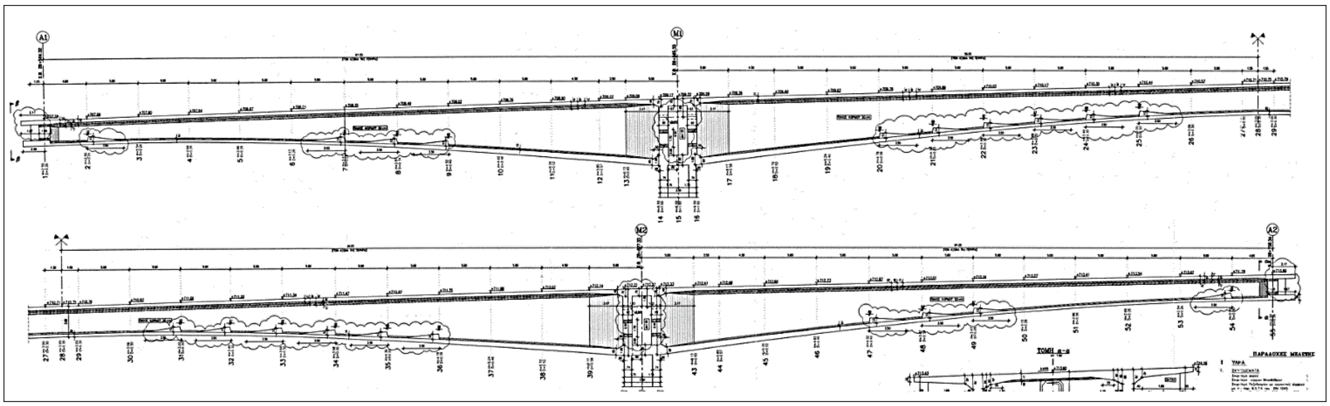

Fig. 4 – Detailed longitudinal section of the deck with tendon anchorage heads.

At the abutments, the deck is supported vertically by pairs of pot bearings (Fig. 5), that allow the horizontal movements via low friction PTFE (Teflon) coated steel plate interfaces in order for it to be free to thermally expand longitudinally and rotate around the vertical axis, whereas the transverse displacements are constrained via RC shear keys fixed on the abutments. Thick transverse RC diaphragms enforce the pier to deck (Fig. 6) and the pots to deck (Fig.5) connections, smoothly distributing the pier and pot forces to the deck section and thus avoiding buckling of the deck box sections due to high local stress concentration.

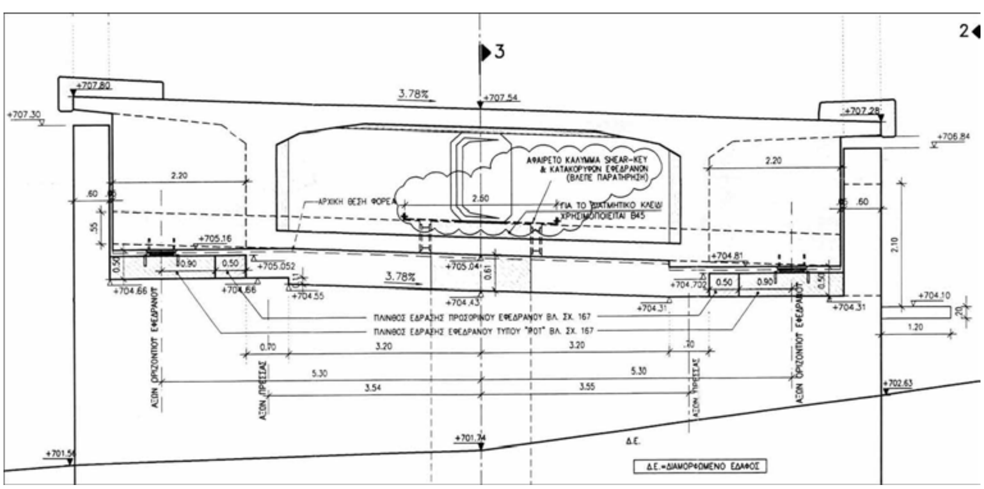

Fig. 5 – Section at the abutments illustrating the pot bearings the shear key and the end cap.

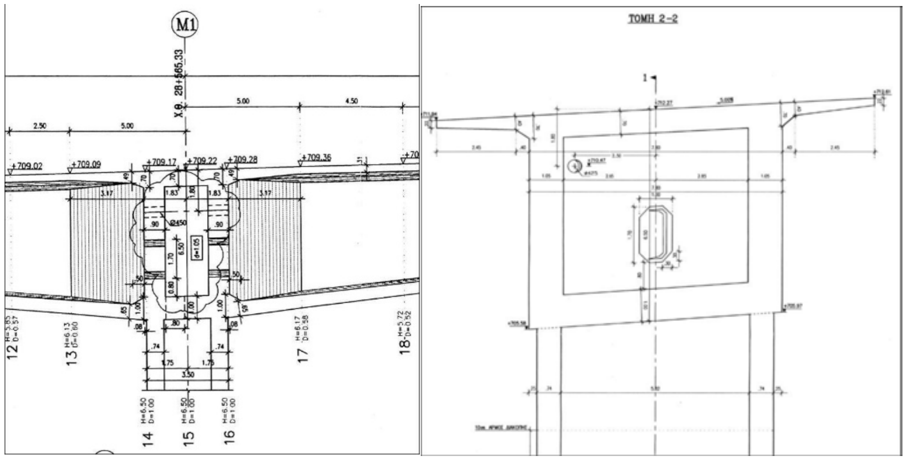

Fig. 6 –Longitudinal (left) and transverse (right) sections at the pier to deck connection.

3. The Finite Element Model

For the creation of the FE model the SAP2000 V16 structural analysis software was used. In order to minimize the geometrical discrepancies between the model and the actual structure the box girder deck was modelled using four node homogeneous shell finite elements that utilize the Mindlin –Reissner bending theory (thick shell)[1].That way it was made possible to simulate the variations of the boxes height and the walls thickness in the longitudinal direction that greatly affect the stiffness distribution and consequently the dynamic behavior of the structure (Figs 7&8).On the contrary for the various thickness changes and haunches of the box sections in the transverse direction, the approach of equal areas of the variable-thickness walls and the constant thickness elements that simulate them was followed (Fig. 9).

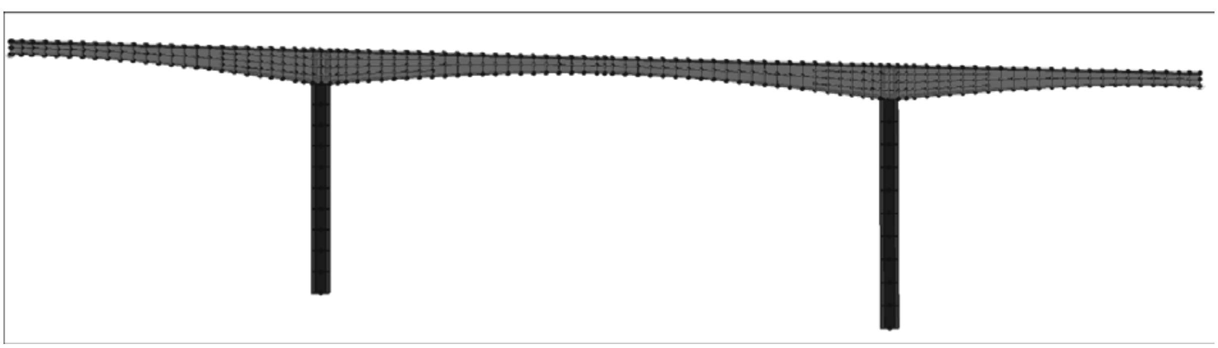

Fig. 7 – Vertical view of the FE model.

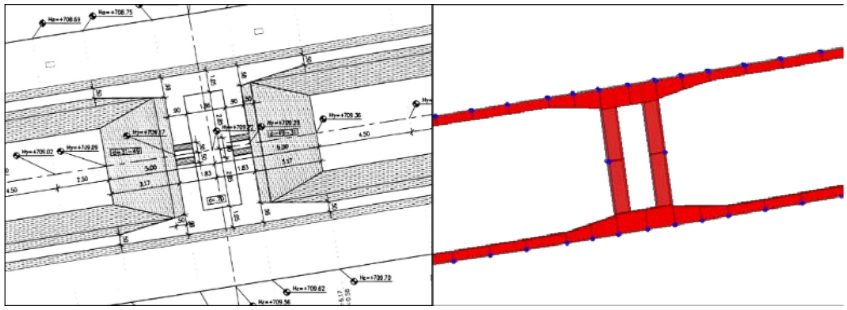

Fig. 8 – Horizontal section at the pier to deck connection (left) and corresponding FE connectivity (right).

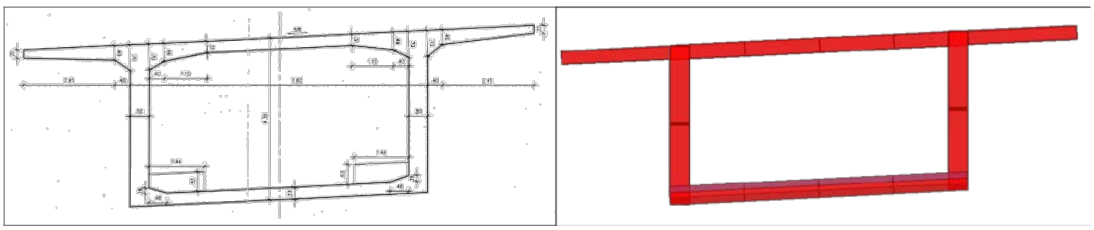

Fig. 9 – Typical deck's box section and corresponding FE model simulation.

The rectangular hollow section piers were modeled using beam finite elements. The connection of the beam elements of the piers to the shell elements of the deck has been modeled using body constraints. These constraints were defined appropriately in order to adequately represent the physical connectivity of the real structure[2] (Fig.10).

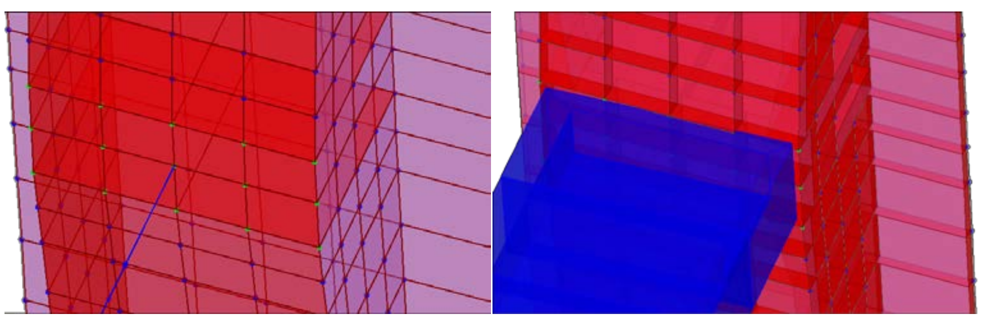

Fig. 10 – Beam FE (blue) connection of pier to shells Fes (red) of the deck. In green the nodes of the body constraint.

The transverse RC walls (end caps) that connect the pot bearings and the shear key with the decks end sections at the abutments were not modeled. Instead, due to their very high in-plane stiffness and the low length to thickness ratio the end caps were considered rigid for the in section part and were substituted by body constraints (Figs. 5&11). The bending flexibility of the parts of the end caps that connect the pot bearings with the deck was substituted by translational springs in the longitudinal direction at the pots' locations. Similarly in order to account for the flexibility of the piers foundation system, rotational springs were inserted at the ground nodes of the piers.

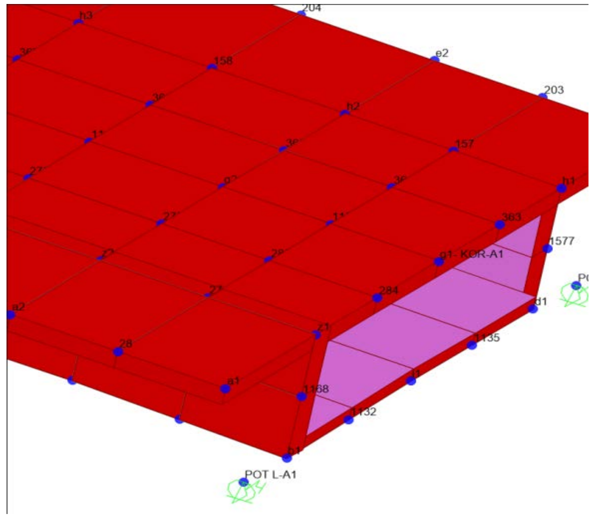

Fig. 11 – Substitution of end cap with body constraint including end nodes and pot-bearings nodes.

The amount and distribution of the mass on a structure defines together with the stiffness the dynamic characteristics of the system. The masses that were accounted for in the analyses were the mass of the deck and the piers, the masses of the tendons anchorage heads and the masses of the road layers and sidewalks. The first ones were inserted as material mass densities and were automatically integrated and assigned to the models nodes by the software, whilst the latter two were inserted as area loads and automatically converted to masses. The magnitudes and distribution of these area loads were modeled according to the information provided from the design plans of the bridge.

4. Finite Element Model updating based on identified modal data

A model updating procedure can be described as the process during which various parameters that define a theoretical or numerical model of a physical system are tuned in order for it to predict the measured behavior of the physical system as accurately as possible. Generally in the case of structural dynamics the models are finite element models that predict the dynamic behavior of the corresponding real structures and the goal is in the form of matching the modal quantities (eigenfrequencies and eigenvectors) predicted by the model to the actual modal quantities of the structure. Various techniques have been developed for the identification of the modal quantities of the real structure [3], [4]. These are estimated by analyzing the responses of the structure under known or quasi-known excitations. These procedures produce quantitative information for the structures eigenfrequencies and eigenvectors of some of its vibration modes. The types and the accuracy of the identified modes are strongly dependent on the characteristics of the excitation during the measurements, the number and the location of the measured degrees of freedom, the signal to noise ratio of the recordings which in turn depends on the strength of the excitation, and the degree of nonlinearity in the structures response.

In the present case the identified quantities came only in the form of 8 eigenfrequencies, the qualitative estimation of their respective mode types and an estimation of their modal damping factors. An output-only identification method[5] was used that assumes that the excitation had white-noise characteristics and in this case was provided by the light traffic conditions at the time of the measurements.

In table 1 the identified quantities are presented as they were provided. In the first row the identified modes description is given, in the second the eigenfrequencies, the third row gives a measure of fit error (the lower the better) and the fourth gives an estimation of the modal damping ratio. The different values in every cell correspond to equal runs of the identification algorithm and as these increase and agree with each other so does the reliability of the estimation for a particular mode. Taking that into account it is apparent that the identified eigenfrequencies of the vertical bending modes were significantly more reliable than the horizontal ones, as these were the modes that were sufficiently excited by the light traffic conditions during the measurements.

Table 1 – Modal identification data as provided by Egnatia Odos S.A.

No	Identified frequencies	f_i (Hz)	fit error measure	Identified Modal damping ζ (%)
1	*1st Bending Horizontal*	0.887/0.885/0.874/0.878/0.881/**0.885**/0.884/0.885/0.884	18.7/12.1/12.8/13.4/19.4/**11.7**/17.1/12.1/15.6	10.90/4.3/3.03/2.71/**3.62**/3.51/4.3/4.11
2	*1st Bending vertical*	1.46/1.42/1.43/1.43/1.43/1.43/1.44/1.42/1.42/**1.42**/1.42/1.42	50/1.18/19.4/0.94/0.78/1.06/?/1.62/1.29/**0.81**/1.18/1.51	2.23/2.2/2.26/3.01/1.87/1.82/2.24/1.9/1.77/**2.02**/2.2/2.14
3	*2nd Bending Horizontal*	2.03/2.03	13.7/9.81	4.70/4.07
4	*inconclusive*	2.32	9.46	9.61
5	*2nd Bending vertical*	2.76/2.71/2.75/2.71/2.72/2.73/2.73/2.73/2.72/2.7/2.71/**2.71**	41/1.85/?/?/1.61/0.515/1.53/1.77/6.92/6.85/7.54/**1.54**	2.49/?/?/?/4.68/1.39/1.64/1.31/1.43/1.8/2.06/1.85/**1.88**
6	*3rd Bending Horizontal*	3.17	47.1	1.67
7	*3rd Bending Vertical*	3.46/3.43/**3.44**/3.44/3.45/3.47/3.46/3.46/3.43/3.43	4.19/3.25/**3.13/**4.13/4.73/3.47/9.3/8.67	1.17/1.27/**0.88**/1.27/1.43/1.19/1.64/1.64
8	*4th Bending Horizontal.*	3.89	13.3	2.57
9	*4th bending Vertical*	5.02/4.47/**4.49**/4.47/4.48/4.42/4.47/4.47	14.5/1.44/**0.64**/1.48/2.29/1.84/2.24/1.91	1.92/0.885/**1.17**/0.775/0.561 1.3/1.56/1.61

The model parameters chosen to be updated were the elastic modulus of the decks concrete (E_d), the elastic modulus of the piers concrete (E_p), the rotational flexibility of the piers foundation system (k_r), the flexibility of the links between the pot bearings and the deck at the abutments (k_u), and the effect of the steel tendon sections in the longitudinal stiffness of the top and bottom horizontal walls of the decks box section (f_l). Because of the relatively young age of the structure, the implementation of modern construction techniques and the fact that it hasn't endured any significant damaging incident (quake, collision, settlement); the absence of significant geometrical deviations and localized damage was assumed and so, these factors were not considered for updating.

In table 2 the initial values of the updating parameters is shown. The two moduli of elasticity were chosen according to their classes[6] (B45 for the deck and B35 for the piers) the initial flexibility of the foundation due to lack of specific data was taken as infinite (full fixity) and so did the pot links flexibility. Also the effect of the tendons was initially neglected.

Table 2 – Initial values of the to-be-updated model parameters.

	E_d (GPa)	E_p (GPa)	k_r(kNm/rad)	k_u (kN/m)	f_l (ratio)
Initial values	37.5	34.5	Fixed	Fixed	1.0

Because of the low quality of some of the identification data, a feedback step between the model results and the identification results was conducted. Table 3 shows both the, as-given identified and the FE models modal data with initial parameters values. From the given identified data, mode 4 was discarded as unreliable due to absence of type description and unrealistically high damping ratio and mode 6 was discarded due to high fit error and because of its absence in the model analysis results for every possible values combination of the updated parameters (see also table 1). The comparison between the selected-as-reliable identified modal data and the corresponding calculated form the FE model for initial parameter values are shown in table 4 and in figure 12. Figure 13 shows the corresponding deviations in %.

Table 3 – Identified *as-given* modal data compared to FEM calculated with initial parameter values.

	Mode type description (Identified as given)	Identified eigenfrequencies	FE model eigenfrequences	Mode type description (FE model)
		(Hz)	(Hz)	
1	1st Bending Horizontal	0.885	0.936	1st Bending Horizontal
2	1st Bending Vertical	1.42	1.285	1st Bending Vertical
3	2nd Bending Horizontal	2.03	1.952	2nd Bending Horizontal
4	inconclusive	2.32	2.333	2nd Bending Vertical
5	2nd Bending Vertical	2.71	3.240	3rd Bending Vertical
6	3rd Bending Horizontal	3.17	3.567	3rd Bending Horizontal
7	3rd Bending Vertical	3.44	3.727	4th Bending Vertical
8	4th Bending Horizontal.	3.89	4.972	1st Deck Torsional
9	4th bending Vertical	4.49	5.204	Higher order mixed type

Table 4 – Selected *as-reliable* identified modal data compared to FEM calculated with initial parameters.

	Mode type description (identification)	Identified eigenfrequencies	FE model eigenfrequences	Mode type description (FE model, initial)	deviation
		(Hz)	(Hz)		
1	1st Bending Horizontal	0.885	0.936	1st Bending Horizontal	5.45%
2	1st Bending Vertical	1.42	1.285	1st Bending Vertical	-10.51%
3	2nd Bending Horizontal	2.03	1.952	2nd Bending Horizontal	-4.00%
4	2nd Bending Vertical	2.71	2.333	2nd Bending Vertical	-16.16%
5	3rd Bending Vertical	3.44	3.240	3rd Bending Vertical	-6.17%
6	4th Bending Horizontal.	3.89	3.567	3rd Bending Horizontal	-9.06%
7	4th bending Vertical	4.49	3.727	4th bending Vertical	-20.47%

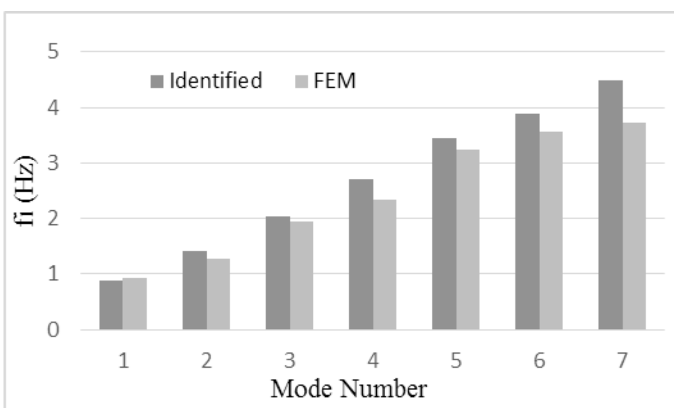

Fig. 12 – Identified vs. calculated eigenfrequencies values with initial model parameters.

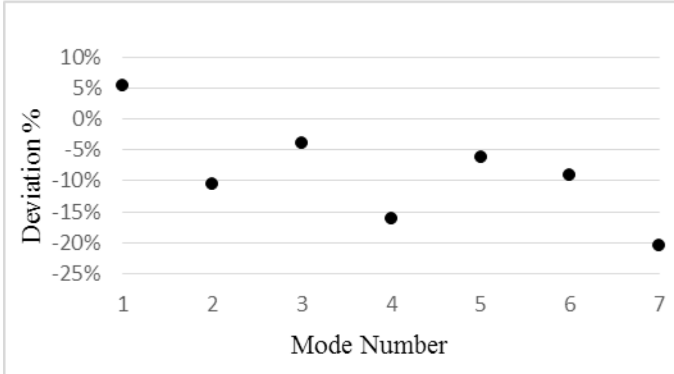

Fig. 13 – Identified vs. calculated eigenfrequencies deviations with initial model parameters.

The model with the initial parameters appears to be generally more flexible than the actual structure, whilst there are also significant differences between the deviations of the individual eigenfrequencies. This was an indication of the fact that apart from the increase of the general stiffness of the model through the global stiffness parameters (E_d, E_p), the updating should include some parameters that selectively affect some modes differently than others, hence the inclusion of k_u and f_l.

Table 5 shows the results of a rough sensitivity analysis in order to get some insight on the effect of the updating parameters to the eigenfrequency values of the model. As expected a small change in the values of either piers (E_p) or decks' (E_d) elastic moduli alter the eigenfrequencies values in generally the same manner for all modes with E_d influence showing a slightly increased preference for the modes of higher order and the E_p for the ones of lower order. The foundation system flexibility (k_r) is shown to affect mostly the lower order modes whilst the flexibility of the pot-to-deck links (k_l) affects only the horizontal bending modes.

Table 5 – Sensitivity analysis of the eigenfrequencies to the models updating parameters.

Mode	Ed decrease -20%	Ep decrease -20%	fl increase 10%	kr decrease (-1%)	kl decrease (-0.1%)
	Differ.	Differ.	Differ.	Differ.	Differ.
1st Bending Horizontal	-7%	-4%	2%	-3%	-7%
1st Bending vertical	-8%	-3%	2%	-2%	0%
2nd Bending Horizontal	-10%	-2%	2%	-2%	-9%
2nd Bending vertical	-9%	-2%	3%	-1%	0%
3rd Bending Vertical	-11%	-1%	3%	-1%	-1%
4th Bending Horizontal.	-11%	-1%	2%	-1%	-8%
4th bending Vertical	-11%	-1%	3%	-1%	-1%

Taking all the above into account an empirical trial and error model updating procedure was conducted with its results frequency-wise presented in table 6 and figure 14. The procedure achieved a satisfying level of conversion, roughly +-6%, between the identified and the calculated eigenfrequencies. The final values of the updated parameters are shown in table 7, along with the initial ones.

Table 6 – Comparison of the identified eigenfrequencies to the calculated ones of the updated model.

	Mode type description	Identified eigenfrequencies	FE model eigenfrequences	deviation
		(Hz)	(Hz)	
1	1st Bending Horizontal	0.885	0.879	-0.64%
2	1st Bending vertical	1.42	1.434	1.00%
3	2nd Bending Horizontal	2.03	1.910	-6.28%
4	2nd Bending Vertical	2.71	2.615	-3.65%
5	3rd Bending Vertical	3.44	3.655	5.88%
6	4th Bending Horizontal.	3.89	3.721	-4.54%
7	4th bending Vertical	4.49	4.219	-6.42%

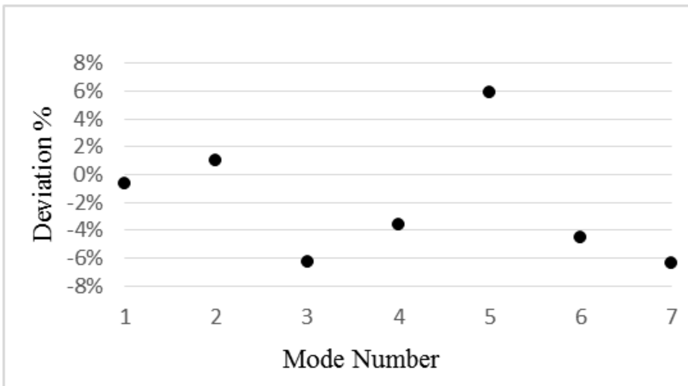

Fig. 14 - Identified vs. calculated eigenfrequencies deviations with updated model parameters.

Table 7 – Initial and final values of updated model parameters.

	E_d (GPa)	E_p (GPa)	k_r(kNm/rad)	k_u(kN/m)	f_l (ratio)
Initial values	37.5	34.5	Fixed (10^{11})	Fixed	1
Final values	48.0	36.0	$3*10^8$	$8*10^6$	1.10
Difference %	+28%	+4.3%	-	-	+10%

The final values of the updated parameters are within reason. Although an increased final elastic modulus of concrete compared to the nominal values is a common occurrence particularly in prestressed concrete structures [7], further justification of the particular updated values for E_d and E_p in not feasible due to lack of more specific data. The spring constant of the plates that link the pot bearings to the deck is also reasonable and comparable with the one obtained by a sub-model analysis which resulted in an equivalent $k_{u,\ sub} = 6*10^6$.

5. Remarks-Conclusions

Upon updating the FE model, an adequate frequency-wise level of convergence was achieved. In absence of identified eigenvector data, several assumptions based on engineering judgement had to be made in order to proceed but the level of convergence and the reasonable final values of the updated parameters were indicative of their validity. Nevertheless, one should adopt the products of the updating with caution as the procedure was not exact and there could be infinite combinations that would produce similar eigenfrequencies.

The actual stiffness of the structure was found to be increased compared to the nominal. One could safely assume that this was due to the increased stiffness moduli of the piers and mostly of the prestressed box girder decks' which is a usual case for prestressed concrete. Significant also proved to be the influence of the pot-to-deck walls flexibility in order to increase the vertical bending modes while keeping in control the horizontal ones.

A higher than expected stiffness is usually beneficial under static loading conditions as it reduces static displacements, but this is not the case with dynamic loading such as an earthquake, due to the nature of which a higher structural frequency can increase significantly the seismic energy intake especially in the longer periods which is usually the case with long bridge structures.

It should be clarified that the updated linear FE model corresponded to the bridge behavior under the measurement conditions where, because of the light loading, no pier section cracking and no sliding at the pots occurred. Therefore, the eigenfrequencies both identified and calculated, corresponded only to this state and are not at all employable for assessing the actual seismic behavior of the bridge. That of course does not impair the validity of the results of the updating procedure as the updated parameters are largely independent of eider the magnitude of vibrations or the nonlinearities of the structure.

Finally, as a general comment on the updating procedure it could be said that the various model updating procedures are most valuable when they are based on full and reliable identification data. When this is not the case, the incomplete data could still bear valuable information but should be used with robust engineering reasoning and in a reciprocal way with the results of a meticulous and finely defined mathematical model.

6. References

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