Numerical and operational modal analyses of the “Ponte delle Torri”, Spoleto, Italy

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ABSTRACT: The *Ponte delle Torri* is a large historical construction that connects Colle Sant’Elia with Mount Monteluco in Spoleto, Italy. It was probably built in the XIII century, possibly on Etruscan or Roman ruins. The bridge superstructure is made up of a pedestrian deck, provided with a water canal on one side, supported by lancet arcades and stone piers known as “towers”. As testified by recent documentation and on-site inspections, the bridge’s state of damage is rather widespread and extended, urging local and national institutions to start suitable structural monitoring activities, which were included within the RoMA (Resilience enhancement of a Metropolitan Area) project. The project comprises many monitoring techniques (high-resolution photogrammetry, 3D laser scanning, FBG monitoring, satellite InSAR, etc.) to assess the bridge cracks pattern and its structural health by a multidisciplinary approach that permits mutual validation. In particular, the present paper describes the numerical and operational modal analyses performed for the study of the structure and the methodology for the validation of the implemented Finite Element Models (FEMs). As a first step for the study of the bridge, the modal analysis of a preliminary FEM was executed in order to have an idea of the dynamic behavior of the structure and to guide the design of the on-site sensors positions. In such positions ambient noise was measured with velocimeters. Then, more refined FEMs were constructed and validated. In particular, mono-, bi- e tri-dimensional FEMs were generated through different numerical techniques and their resulting modal analyses were compared with the results of Operational Modal Analysis (OMA) techniques applied to the recorded velocimeters data.

1 INTRODUCTION

The masonry structures represent a remarkable testing ground not only for the technicians that commonly work on the urban context, but also for researchers who have been involved for years trying to find a strategy of analysis about how to "standardize" the calculation procedures such as reinforced concrete and steel constructions. Recent seismic events occurred in Umbria and in Emilia helped to increase the interest to study the behavior of ancient masonry buildings and to design heavy interventions of recovery for the existing assets.

The structural behavior of masonry buildings is generally very complex for the inhomogeneity of the material constituting the masonry, the lack of structural integrity, the different types of wall hangings.

Moreover, the evolution of historical events through which the building was transformed adds new uncertainties to the modeling.

For all these reasons the modeling of historic masonry structures is a complex problem because of the difficulty of properly considering the geometry, materials and boundary conditions.

The discrete nature of the masonry (mortar and bricks) pushes towards the use of modeling techniques that allow homogenizing the building materials so that the numerical simulations can be conducted considering an equivalent continuous medium. In fact, although the available computing capacity is significant nowadays, it is still very challenging to approach this problem with a discrete analysis, because the number of variables to address would be extremely high.

In the present paper, several types of Finite Element Models (FEMs), supported in the Italian Guidelines for the assessment and mitigation of seismic risk of the Cultural Heritage (D.P.C.M. 2011), were used to characterize the behavior of an historic bridge known as “*Ponte delle Torri*”, located in Spoleto, Italy.

The results presented in this paper were developed within the RoMA (“Resilience enhancement of a Metropolitan Area”) project (through an agreement between ENEA and the Italian Ministry of Cultural Heritage and Activities), funded by the Italian Ministry of Education, Universities and Research for enhancing the resilience of strategic assets in urban areas, such as cultural heritage assets, also through the integration of the widest variety of monitoring techniques able to provide a more complete understanding of ancient structures (De Canio et al. 2015).

The procedure applied is based on the combined use of finite element modelling and in situ environmental experimental testing. In particular, part of the experimental results have been used to calibrate and validate the finite element model.

2 THE "*PONTE DELLE TORRI*" OF SPOLETO

2.1 *History and main characteristics*

The "*Ponte delle Torri*" is a huge ten-arcade masonry infrastructure that connects two hills, Colle Sant' Elia and Monteluco. In ancient times, it used to be an aqueduct, probably built in the 13th century on previous structures exploited as foundation, and a bridge at the same time. It is constituted by a roadway and a water canal in which the water used to flow exploiting the slope of the artificial track, located on top of a continuous wall prolonging the height of the parapet on the south side of the roadway. The deck is supported by ogive arches on stone piers, historically named "Torri", or towers in English (Figure 1). The water network has undergone several changes: in 1891, the "*Ponte delle Torri*" was a key element of the water system of Spoleto and was used as a road link between Monteluco and Colle Sant'Elia. The waters came to the bridge through three pipelines, and, before being transported to the city, served as the driving force of two mills.

Figure 1. General view of the Tower Bridge of Spoleto.

2.2 *The bridge structure*

The bridge has a stone structure with shoulders and piers ("towers") made up of rubble walls in a square matrix assembled by mortar and lime. The structure is only apparently regular. In fact, piers and arches have all different shapes and sizes, as well as the walls towards Colle Sant'Elia and Monteluco are also diverse in the masonry texture, due to their different construction timing and subsequent rebuilding interventions. The piers differ from each other even in their development in height: piers closer to Monteluco are more massive than the others and are reinforced by arches raised about half of their height, while clear spans between piers closer to Colle Sant'Elia are longer than the others (Figure 2).

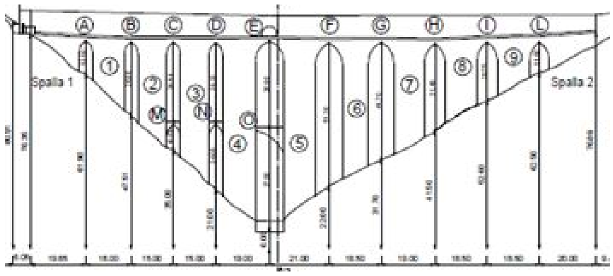

Figure 2. Longitudinal section of the bridge from south. Piers are number from West to East.

With reference to the side of the "Rifolta", and in particular in the lower portions of the piers, it is built as "booklet masonry", while, on the side of the Rocca it is built as well-arranged masonry stones, characterized by perfectly vertical corners made with squared elements often well linked and interconnected by a thin layer of mortar.

The towers are joined at the top by arches made up of rather squared stone blocks.

The wall bearing water canal has a thickness of about 2.00 m, while its height is variable (greater towards Monteluco and smaller towards Colle Sant'Elia). The bridge has an overall length of about 230 m, while the highest tower is over 70 m (Sansi 1984).

2.3 *Analysis of the current state*

The current state of damage, and more particularly the cracks in the bridge, are closely related to the time of construction, the quality of the materials, as well as to subsequent structural interventions related to its architectural modifications and functional changes.

The bridge has a state of damage typical of historic masonry bridges. At the top of the arches, especially those on the slope of Monteluco, there are heavy water infiltrations resulting in losses of mortar binder and wall apparatus skiving. This is accompanied by the formation of cracks and peeling of the outer frames at the intrados of the arches. A similar state was also surveyed at the arches of the deck, showing biotic aggression of unexposed areas and serious deterioration of materials in the walls. The same type of damage was detected at the arches on the side of the Colle Sant'Elia. Moreover, a widespread damaged layer, especially in the lower part of the piers, was also detected. This has caused the expulsion of some cornerstones of the piers, the disarticulation of the masonry texture and the collapse of portions of the outer layers.

However, the pier number 4 (Figure 2) shows the most severe crack pattern: two very large cracks are clearly visible (Coccetta et al. 2013).

Confirming different period of construction, the material of the stones and masonry texture are different depending on the different portions of the same pier. On the top, close to the arc, the situation improves and the mortar has a good consistency.

The piers on the Colle Sant'Elia side have a better state of preservation: the condition of the masonry texture presents, moreover, different characteristics in the development in height, the clamping edge is also good.

3 THE FINITE ELEMENT ANALYSIS

Modal analyses by different numerical codes were performed for the whole bridge in order to reproduce the first modes in terms of shapes and frequencies and to identify the most interesting locations where positioning the velocimeters for in situ experimental environmental measures.

Different FEMs (1D, 2D and 3D) were also defined to investigate the limitations of 1D and 2D analysis in the transversal effects.

All materials were assumed to work in the linear range adopting a linear elastic model.

The mechanical properties of the masonry were the same for all the FEMs. It is assumed a homogeneous isotropic elastic material with the following mechanical properties (Cluni et al. 2004): Young's modulus $E = 8000\ N/mm^2$, Poisson's ratio $\nu = 0.3$ and density $\rho = 2067\ kg/m^3$.

Perfectly clamped boundary conditions at the base of the piles were considered in the following analyses.

3.1 *FE modal analysis results*

The first four modal frequencies obtained by NASTRAN and FEAP numerical codes with different types of 2D and 3D elements (Table 1) showed very limited discrepancies.

Table 1. Modal frequencies of the first 4 modes by FE analyses.

FEM type	FEAP		NASTRAN	
	2D-shell f (Hz)	3D-brick f (Hz)	2D-shell f (Hz)	3D-hexa8 f (Hz)
Mode 1	0.609	0.601	0.582	0.633
Mode 2	1.050	1.031	1.086	1.162
Mode 3	1.510	1.481	1.520	1.527
Mode 4	1.872	1.864	1.892	1.913

Also the results in terms of modal shapes were very similar with all the applied FEMs. In the Figures 3-4 the modal shapes calculated by FEAP with 3D-brick elements are illustrated, while the top views of the first two modal shapes with NASTRAN 2D-shell elements are displayed in Figure 5.

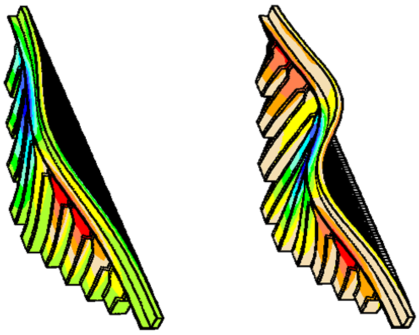

Figure 3. Modal shapes from FEAP (3D-brick): Mode 1 (left) and Mode 2 (right).

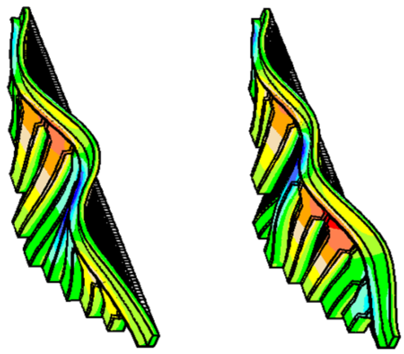

Figure 4. Modal shapes from FEAP (3D-brick): Mode 3 (left) and Mode 4 (right).

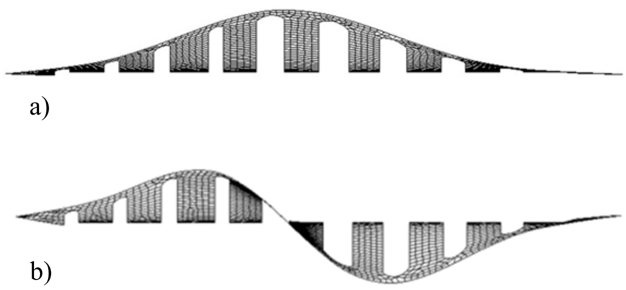

Figure 5. Top view of modal shapes from NASTRAN (2D-shell): Mode 1 (a) and Mode 2 (b).

3.2 *FE linear and nonlinear dynamic analysis results*

Linear and nonlinear dynamic analyses were also performed. In particular, a nonlinear analysis by FEAP numerical code with 8-node brick elements was carried out considering a constitutive law based on a damage model developed by Addessi & Sacco 2015. In such analyses the accelerogram recorded in East direction during the Valnerina earthquake (Figure 6) occurred on 19th September 1979 (M_s = 5.5) was adopted as seismic input in direction transversal to the bridge deck.

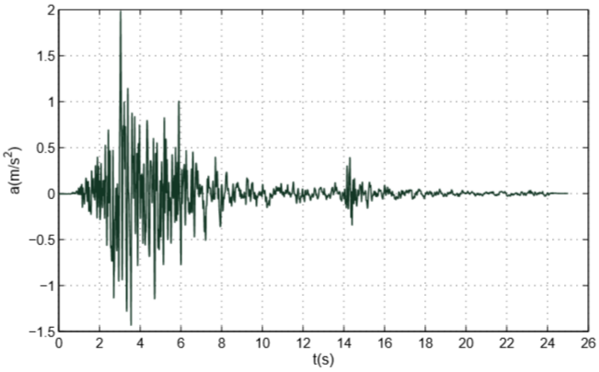

Figure 6. Seismic input accelerogram: Valnerina earthquake of September 19th, 1979 at 21:35:37.

The displacements obtained by linear and non-linear dynamic analysis under seismic action were evaluated and compared for tower 4 (Figure 2).

It is clear that the displacement response in non-linear range, after the first crack, shows a phase shift and a reduction of the peak value with respect to the response by linear simulation (Figure 7). Such shift is greater during the evolution of the damage, and then decreases after reaching the maximum level of damage.

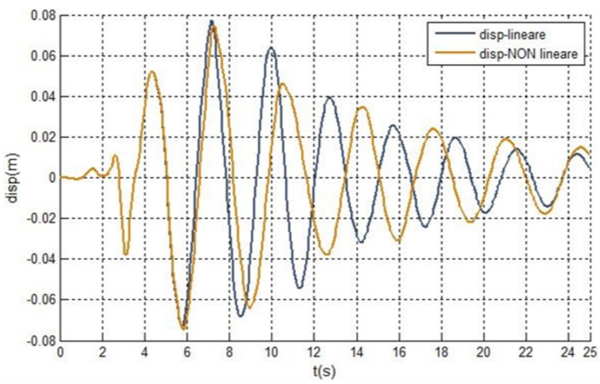

Figure 7. Simulated displacements time-history on top of tower 4 by linear (blue) and nonlinear (yellow) dynamic analyses.

4 IDENTIFICATION OF NATURAL RESPONSE THROUGH OMA

In order to identify the dynamic properties of the studied bridge through experimental data, three seismographs were used to acquire the ambient vibration on site. The used seismographs, SL06 recorders (SARA Instruments) equipped with 0.2 Hz triaxial electrodynamic velocimeters, were set to 200 Hz sampling frequency.

The recorded data were analyzed through OMA techniques, which proved to be very effective and consolidated in identifying the natural response of bridges (Brincker et al. 1996, Araiza Garaygordobil 2004).

4.1 *Test setups*

Multiple Test Setups Measurement Procedures (MTSP) were performed with one common measurement point on top of the central pier of the bridge (velocimeter 104 in Figures 8-9) as reference sensor.

As the reference sensor was staying in the same positions during all setups, it basically measured the mode shapes in this position over and over, while other sensors were moved to different positions on the bridge. Such reference position was determined as the measurement point where the modes of interest were supposed to have the highest response level, according to FEM modal shapes (see previous paragraph 3.1).

Data were acquired in eight configurations, each representing a test setup in the MTSP (see Figures 8-9). All configurations were acquired for at least 20 minutes.

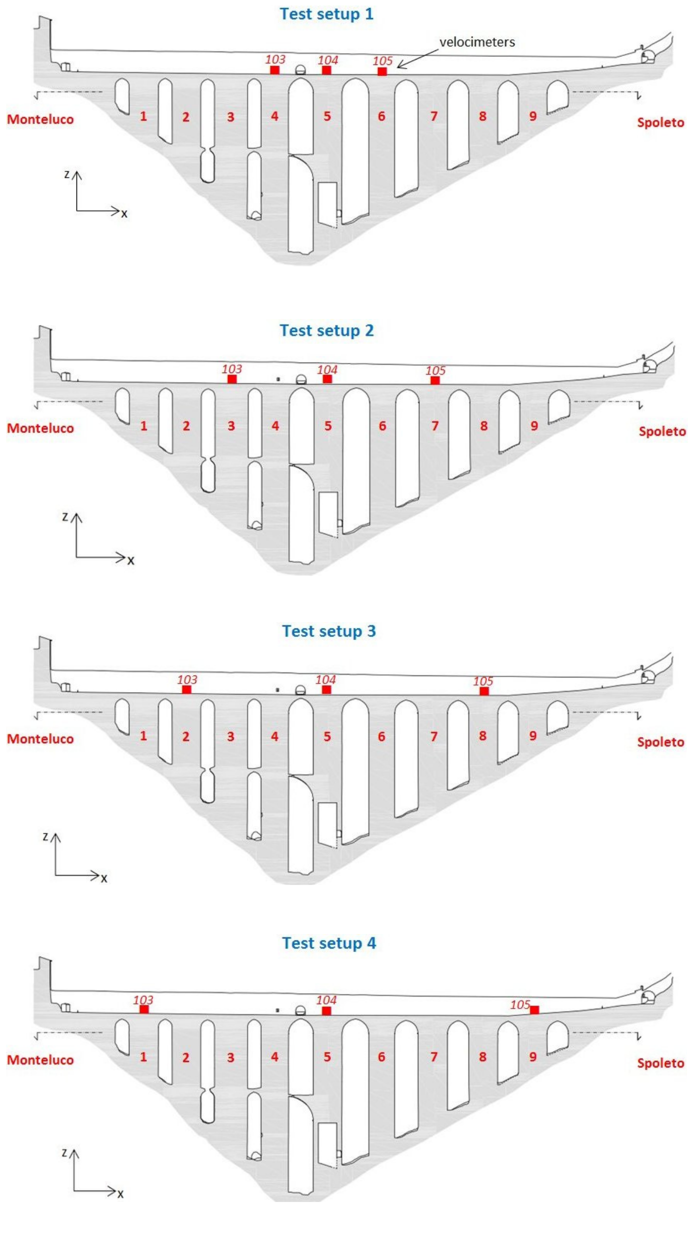

Figure 8. Setups 1-4 used in ambient vibration testing with velocimeters (103, 104 and 105) for OMA analysis

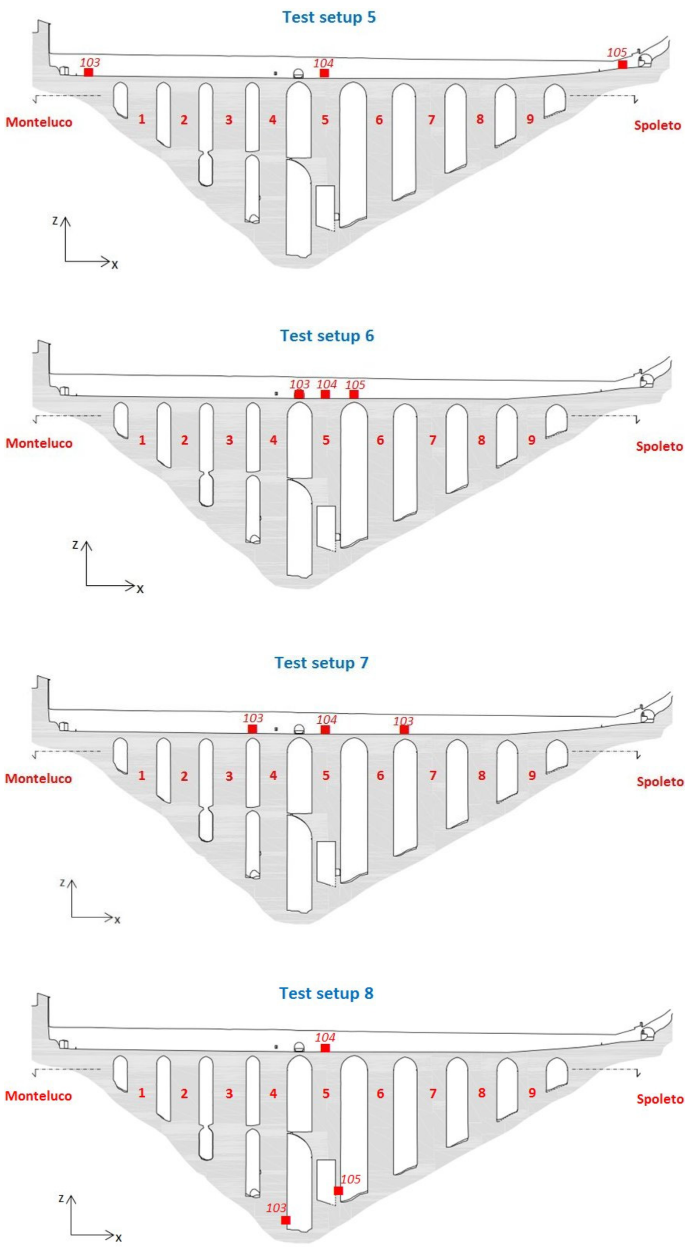

Figure 9. Setups 5-9 used in ambient vibration testing with velocimeters (103, 104 and 105) for OMA analysis.

4.2 *OMA techniques*

The OMA techniques implemented in ARTeMIS Modal Pro software were used for the present study. Among the several techniques available in ARTeMIS Modal Pro the Frequency Domain Decomposition (FDD), the Enhanced Frequency Domain Decomposition (EFDD) and the Stochastic Subspace Identification (SSI) were utilized (Brincker et al. 2001, Peeters & De Roeck 2001).

In particular, in FDD and EFDD modal parameters are estimated by simple Peak Picking (PP), which is a quite manual and subjective procedure, but it is very effective when the modes are well separated. EFDD provides also an estimation of modal damping.

SSI is a more sophisticated and automatic procedure based on time-domain approach.

4.3 *Results of OMA*

The results of the identification of the modal frequencies in the first three modes obtained by applying the OMA techniques are resumed in Table 2.

The modal estimation graphs obatined by OMA techniques are shown in Figures 10-12.

Table 2. Natural frequencies in the first three modes by the applied OMA techniques.

OMA technique	FDD f (Hz)	EFDD f (Hz)	SSI f (Hz)
Mode 1	0.635	0.634	0.635
Mode 2	1.021	0.956	1.018
Mode 3	1.509	1.360	1.504

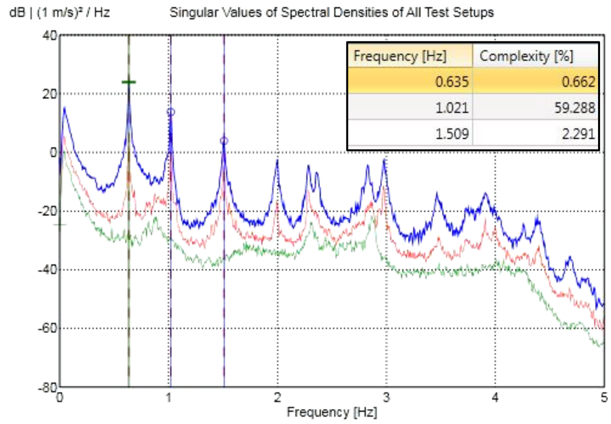

Figure 10. Modal estimation graph by FDD analysis.

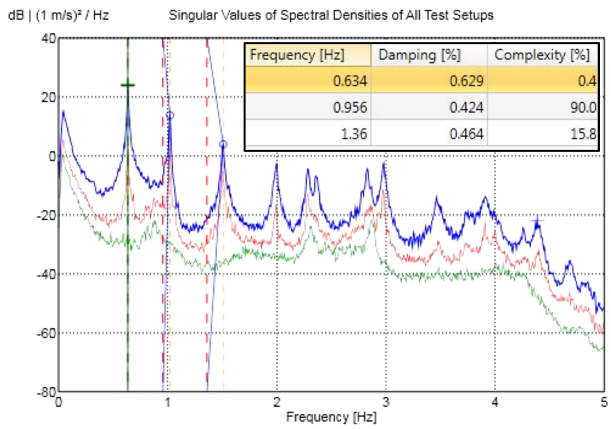

Figure 11. Modal estimation graph by EFDD analysis.

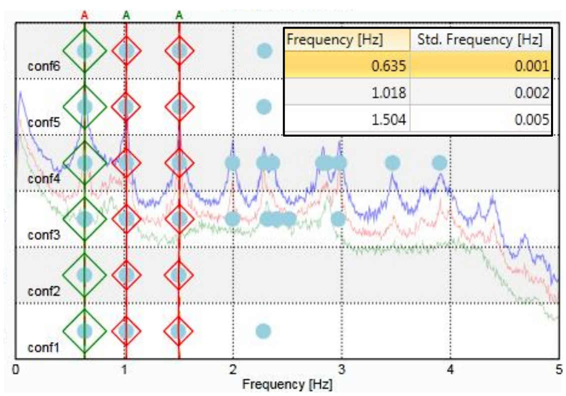

Figure 12. Modal estimation graph by SSI analysis.

4.4 *Comparison between FE and OMA results*

As a confirmation of the validation process of the numerical model of the bridge, the identification of modal parameters using the OMA algorithms showed a good agreement between the environmental data and numerical results.

The first three modal shapes obtained with all applied OMA techniques are very similar to the corresponding modal shapes simulated by FEM analysis.

The modal shapes obtained with FDD for the first three modes are shown in Figure 13. The related three modal frequencies calculated by averaging the three applied OMA techniques are compared in Table 3 with the corresponding frequencies extracted from the two FEMs created with 3D-type elements.

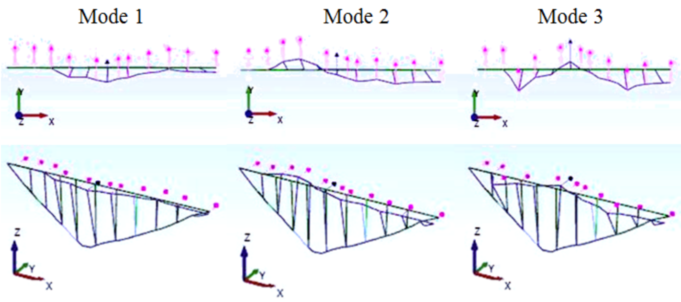

Figure 13. First three modal shapes obtained by FDD analysis.

Table 3. Comparison of the first three modal frequencies by the applied OMA techniques (average) and FEMs.

	OMA (average) f (Hz)	3D FEAP f (Hz)	3D NASTRAN f (Hz)
Mode 1	0.635	0.601	0.633
Mode 2	0.998	1.031	1.162
Mode 3	1.458	1.481	1.527

5 CONCLUSIONS

The identification of the modal parameters of the studied bridge (natural frequencies, damping and modal shapes) were carried out by several OMA techniques and were compared with results from FEA simulations to validate the different built FEMs.

The experimental data allowed calibrating the FEM models, minimizing the difference between experimentally measured and numerically calculated natural frequencies. Consequently, more congruent and near-to-real-condition FEMs were obtained. At the end of the process 3D models by both FEAP and NASTRAN provided very robust and affordable results, especially in terms of modal shapes and frequencies, which were all very coherent with the experimental results of OMA analyses. A very good agreement between experimental and numerical results was achieved, confirming the accuracy of the assumptions made in setting up the finite element model.

Future developments of the RoMA project include the use of further monitoring techniques (PSInSAR remote sensing, laser scanner, drone photogrammetry etc.) to assess the bridge crack pattern and its structural health and will be compared with the results presented in the present paper.

ACKNOWLEDGEMENTS

A special thanks to V. Rosato of ENEA for his contribution to the definition of RoMA project, M. Coccetta of Comune of Spoleto for his institutional support, G. Scatolini and M. Marziani of M&G Engineering Srl for their technical support and documentation, CAI for availability in locating sensors and the Italian State Forestry Corps for vegetation clearing around the studied bridge.

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