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Model assessment of a bridge by load and dynamic tests

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ABSTRACT

The assessment and monitoring of structures through in situ tests, especially dynamic tests, is becoming a common procedure due to the evolution of the instruments and numerical processing of data. However, the use of the data based on the supervised approach, which requires engineering knowledge of the structure is needed before assessing the numerical model that will be the "digital twin" model of the construction for future analysis, monitoring and upgrading of the structure. This process is very complex and requires strong competence in the field of structural engineering, but it allows a real reference for the periodic check of the structure health with a method to also define a solution for its maintenance and upgrade. In this paper, a case study of a PC bridge designed by Riccardo Morandi in 1952–1955 is proposed to apply the process of updating a numerical model by static and dynamic in situ tests. The process based on the two different types of tests was separately conducted to evaluate the efficiency and the difference in the results. The main results of the two types of tests overlap for most features, but specific evidenced aspects that are observed have to be considered.

1. Introduction

Recently, the assessment of structural numerical models by dynamic in situ tests, to elaborate the results of ambient vibration tests (AVT), has spread in the fields of existing construction due to the development of low-cost instruments [1–3] and efficient numerical techniques [4]. In fact, operational modal analysis (OMA) allows a dynamic test to be carried out based only on the output using the ambient signal as input [5].

Currently, dynamic tests are gaining increasing importance and efficiency to calibrate reliable numerical models useful for structural analysis [6–10] because the dynamic test does not require the application of loads and the interruption of the service. Nevertheless, with the intensification of the studies and the evolution of the numerical methods, the reliability of the dynamic identification needs to be further investigated, especially with the aim of applying it for damage detection in monitoring processes [11–12].

It is worth noting that the dynamic parameters could not be sensitive to local phenomena, which can be revealed by static measurements, such as strains and displacements, that are more sensitive to the response in their vicinity and, therefore, are better suited to determine local defects [13–14]. Furthermore, the load test is proposed for a new approach for the safety classification of bridges using the results of periodic tests for evaluating the degradation level and the residual life [15].

The key issue of the structural identification in both cases of dynamic and static tests is a process of model updating for using the experimental results of having a numerical model tailored to the real situation [16–19].

This process of model updating [20] can be developed according to two approaches: automatic unsupervised or engineering/manual supervised.

The engineering/manual approach [21–23] involves selecting and updating model parameters based on engineering knowledge, judgement and experience as well as in-field monitoring data or data from visual examination and material testing. Then, iterative trial-and-error processes are used to refine the model but are always supervised by a structural engineer who considers the physical meaning of the numerical model adjustment. Instead, in the case of automatic modal updating [24–27], the candidate parameters for the updating (such as material properties, and geometrical parameters) and the output of the system (natural frequencies, mode shapes, displacements and so on) are selected and grouped into different vectors. Thus, the objective of the analysis is to minimize the difference or error between these two vectors, adjust the selected parameters of the bridge and improve the correlation between the experimental and analytical model without supervision of

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the real behaviour of the structure. It could give the optimal combination of bridge properties that fit the measured parameters but without any check of the physical meaning.

Each approach has negative and positive aspects. The automatic approach requires the application of a prefixed procedure without checking the engineering meaning of the parameter variation, resulting in a certain efficiency of the model in terms of fitting the experimental results without requiring a long period of study. The reliability of the results is based on the high amount of data. The other requires knowledge of the structures and their typical defects or the influence of various boundary conditions, such as restraints or degradation phenomena. The final fitting of the model is not always perfect, especially if the physical phenomenon that can change the numerical response is not well reproduced in the numerical model, and more time to study the construction is necessary. Conversely, the results are very useful to define the type of intervention that is necessary for the health of the structure because the identification process is at the same type as a diagnostic process.

In this paper, the procedure of structural identification is proposed using both static and dynamic tests but applying the manual procedure for model updating, underlining the various steps that contribute to individuating the real in situ boundary conditions through the variation in the elastic response according to the structural engineering analysis of the construction. In this modern approach, the study allows us to understand the general importance of saving the structural competence of using experimental tests for the assessment of detailed numerical models that are necessary for the analysis of the safety of the construction and the design of a monitoring system.

The bridge considered herein is particularly interesting because the static scheme and the technology adopted by Riccardo Morandi in his design in 1952 was innovative and ambitious at the time, but the construction is still complex, and its analysis requires a detailed finite element model (FEM) that is useful to assess through in situ tests. Furthermore, the real boundary conditions must be defined to better address the future design of strengthening interventions.

A load test and AVT were carried out, and the results were analysed separately to underline the different model updating procedures and the efficiency or disadvantages of each test. The results of both procedures are compared, evidencing good agreement and the peculiarity of each procedure. These studies are not common in the literature because usually only one type of test is considered, and the load test is not used for the model assessment.

2. Static and dynamic tests for structural identification

In recent decades, the use of dynamic tests for the calibration of numerical models and the monitoring of the structure's response has been growing and are supported by technical documents, provisions [28] and studies on the procedure to improve the data processing and instruments. Conversely, the load static test is usually carried out to confirm the resistance capacity and the elastic behaviour of the structure. However, the use of both tests for structural identification with reliable calibration of the model is an opportunity rarely proposed in the literature [17,29–31], albeit it could also be important to design an integrated monitoring system. To understand the different contributions that a load static test and a dynamic test can give to the assessment of the structural behaviour, it is necessary to highlight the response of the structure under these different types of tests and the features that can be detected in the two cases.

First, it is necessary to clarify that both techniques are discussed herein to assess the linear behaviour of the structure even if the load test could also highlight a nonlinear response. Thus, the property that can be assessed considering the linear elastic global behaviour of the structure is its stiffness, which is given by the components in the 3 directions, the vertical one and two orthogonal ones. The uncertainties that influence the stiffness are the geometry, the elastic modulus and the restraints for both types of tests, but those of loads (values and position) and masses must be added for the load and dynamic tests, respectively.

This last aspect has to be better clarified because for the load test, the values and position of the loads can be measured, and the uncertainties depend on the precision of these measures, while the masses in the dynamic test depend on the measurements of geometry and weight of unit volume of the materials. Furthermore, it is worth noting that the stiffness, in both types of tests, is considered in the linear elastic field and is calculated as the effect of elastic modulus, geometry and restraints, but the effect of damage (concrete cover spalling, cracking) could influence the results; therefore, only an engineering approach can allow us to understand the physical phenomenon that is represented in the numerical model as a reduction of the elastic modulus.

The most important difference between the two tests is the limitation of the load test to identify only the vertical stiffness of the bridge because vertical loads are applied, which means that the elastic modulus and the vertical restraints of the deck can be identified. In the case of bridges with a simple supported or continuous deck, the properties of the piers are practically excluded by the vertical response, while their properties are involved in frame structures.

Conversely, dynamic tests are based on the vibration response of the structure in the three directions; therefore, the stiffness of the deck in the horizontal directions can also be identified, and the role of the piers takes a paramount importance in the translational stiffness of the system. Therefore, the elastic modulus of the piers, the horizontal restraints of the deck and the restraint at the base of the piers can be identified [32].

Nevertheless, given the apparent efficiency of the dynamic test, it is necessary to demonstrate that the ambient vibration tests impose low stresses on the restraints; therefore, an effect of the friction involved under traffic or during an earthquake could be annulled.

A synthetic definition of the characteristics that can be assessed by the two types of tests is shown in Fig. 1(a) with a detail of the elastic properties that influence them in Fig. 1(b). However, specific observations are introduced through the case study reported in the following sections.

3. Overview of the bridge

The case study bridge (Fig. 2), erected in Benevento and designed by Riccardo Morandi in the early 1950s, although not so large, has great importance since it connects two parts of the city crossing the creek San Nicola that go through a valley with unstable hillsides.

A detailed analysis of its conceptual design is reported in [33], evidencing a careful process of optimization by adopting an innovative method of construction with precasted segments.

The deck of the San Nicola bridge is made of prestressed concrete cast on site with piers and foundations made of reinforced concrete. The static scheme is a portal frame composed of one main span 80.0 m long and two 20.0 m long cantilevers, as shown in Fig. 3a.

The bridge deck is supported by two piers 9.40 m high; each pier, as shown in Fig. 3c, was realized by eight columns of rectangular section, that are transversally connected at the base and top by a transverse beam; the columns are 40.0 cm thick and have variable width from 1.50 m at the base to 4.00 m at the top.

The piers are linked to the foundations underneath by hinges made of steel rebars to avoid bending moments on the foundation that must also contain the thrust; in fact, the foundations also act as abutments. In Fig. 3b, the details of the hinge at the column base are shown.

During the time, the design of the static scheme with the two cantilevers to avoid the problems of the instable hillsides resulted in a winning choice. In fact, the flood of 2015 in Benevento caused the landslide of the slopes without damaging the bridge. After that, sustaining works were realized by gabions and armed lands, as shown in Fig. 4.

The deck consists of a multicellular caisson whose webs vary in

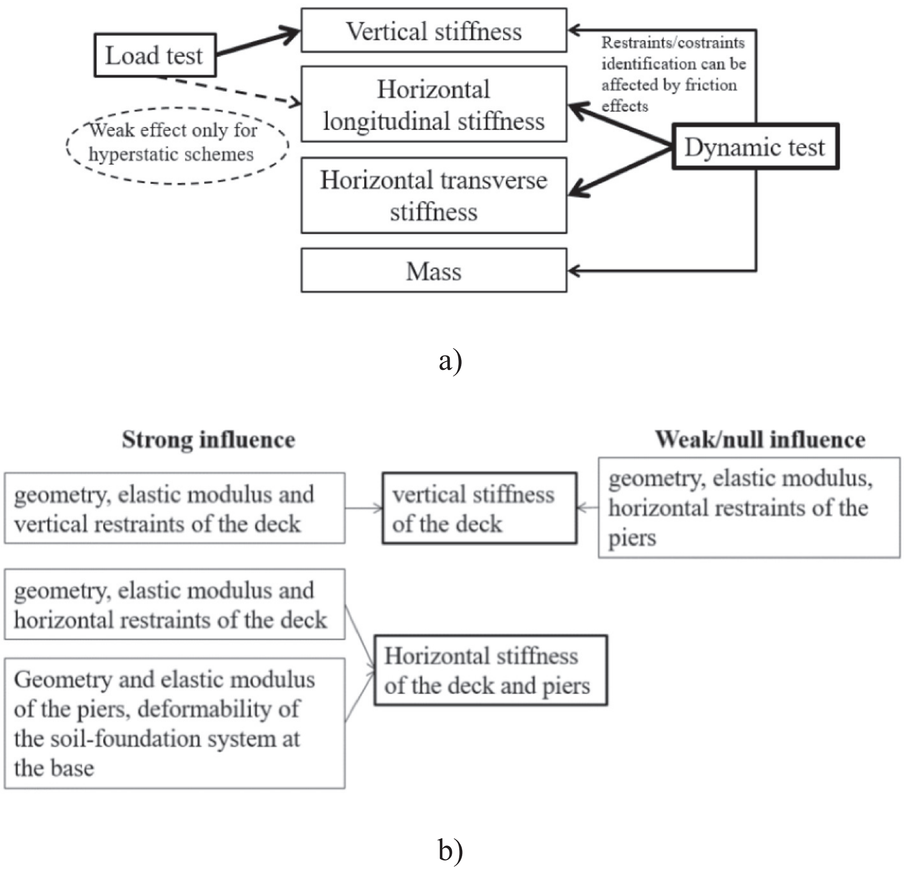


Fig. 1. A) Characteristics of the structure identified by the two types of tests; b) properties that govern the characteristics of the structure.

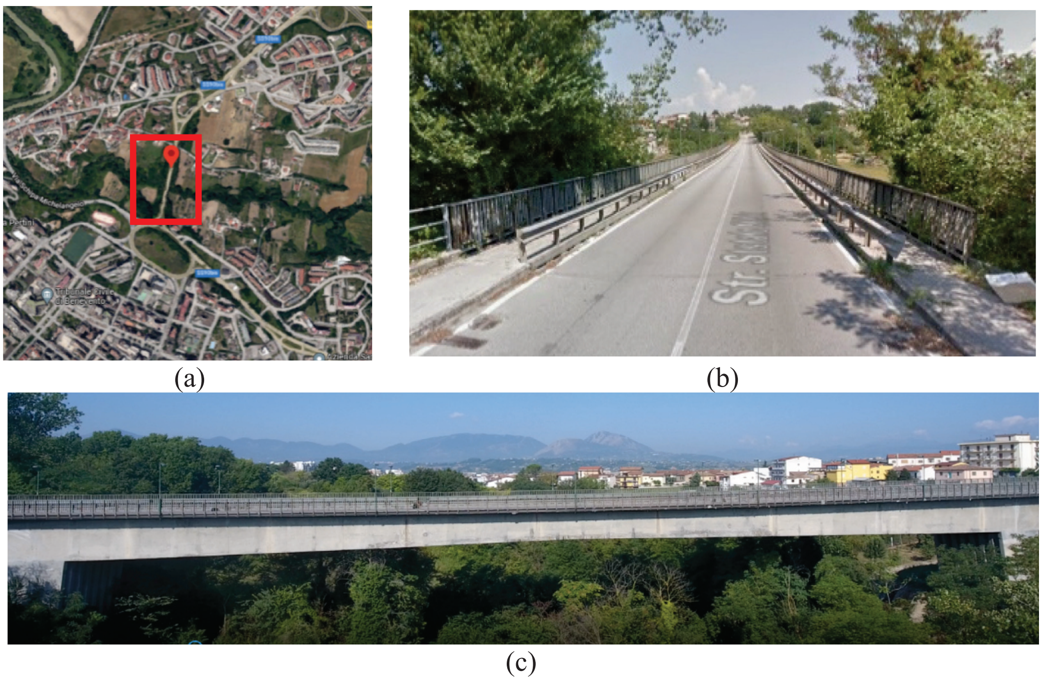


Fig. 2. San Nicola bridge: (a) location; (b) street view and (c) lateral view.

thickness, from a minimum of 13 cm at the midspan to a maximum of 30 cm at the supports (see Fig. 5). Moreover, the deck shows varying depths through the spans (beam with curved intrados), which is characterized by an approximately linear shape, also giving a beneficial arch effect. The depth of the deck cross section varies from a maximum of 3.60 m at the pier supports to a minimum of 1.60 m at the ends of the cantilevers

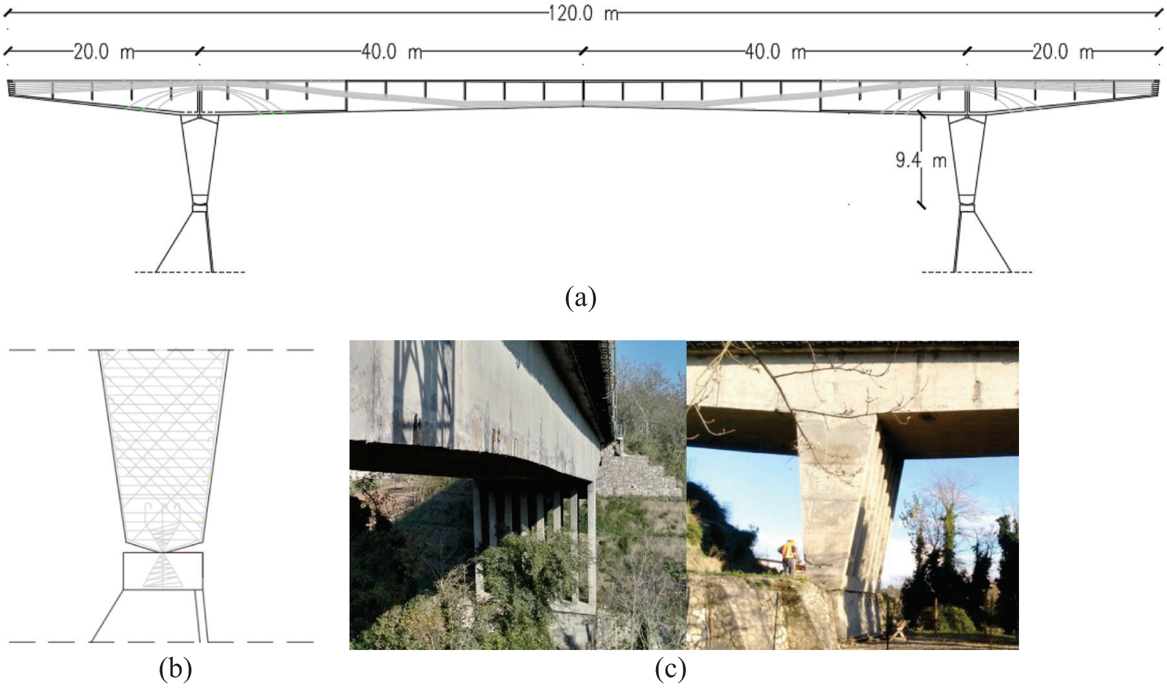


Fig. 3. (a) Longitudinal scheme of the bridge; (b) Detail of the hinge and (c) views of the pier.

Fig. 4. The landslide of the slope in 2015 and the armed lands behind the pier.

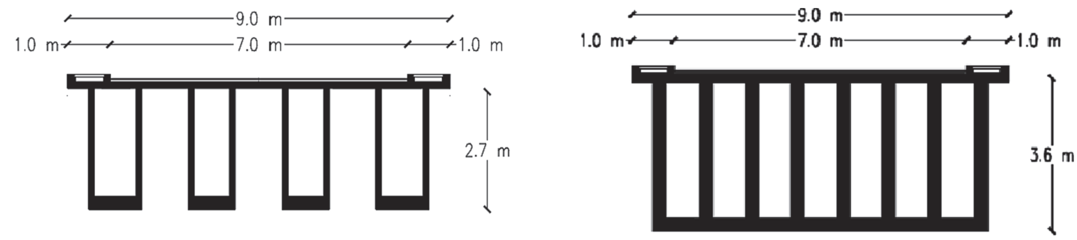


Fig. 5. Cross sections of caissons at the midspan (left) and the supports (right).

and 2.70 m at the midspan.

The deck is characterized by internal tendons that are symmetrical about the midspan. The prestressing steel tendons are composed of 27 aligned wires with a diameter of 5 mm and are placed along a straight line in the top slab where the bending moment shows negative values and in the bottom slab in the midspan but also curved tendons are installed along the entire bridge; the layout of the tendons is reported in Fig. 3a. The thickness of the caissons varies from 13 cm (in the area of the central span where there are no top tendons) to 20 cm.

The upper slab is 9.00 m wide and consists of two traffic lanes, 7.00 m wide, and two sidewalks that are 1.00 m wide, which were added in 1982 using steel profiles.

4. Survey and material characterization

An extensive survey campaign, including surveys, essays and destructive tests on materials, i.e., concrete, steel reinforcement and prestressing steel tendon, was carried out.

The geometric survey of the elements forming the deck and the piers shows that the structure was realized quite as designed.

Several assays were carried out on the deck, highlighting the concrete cover degradation due to the disposal of the water, especially inside the multicellular caisson; however, no cracking is visible. The good conditions of the ducts and tendons were detected except in specific parts where water stagnates. Pier and abutments concrete and reinforcements were also investigated, pointing out a concrete surface degradation but limited corrosion of the steel rebars.

Regarding the materials, 14 cores with a ratio of 1 between height and diameter were taken for the assessment of the concrete compressive strength.

In particular, 9 cores were extracted by the various parts of the deck (slab, webs and transverse of the caisson) e 5 from the piers. The mean cubic compression strength (R_{cm}) was 55.5 MPa for the deck and 40.2 MPa for the piers with a standard deviation of 10.8 MPa and 8.6 MPa, respectively, which is a variation coefficient of approximately 0.20 for both components of the bridge. These results highlight a good homogeneity of concrete in the deck and piers that was realized by different types of concrete. These mean values of the cubic strength were transformed in mean cylinder strength in compression (fcm) equal to 46.1 MPa and 33.8 MPa for the deck and piers, respectively, using a factor of 0.83. These values were used to calculate the elastic modulus adopted in the model described in section 4.

Additionally, samples of prestressing steel tendons were taken and tested in the laboratory, but the elastic modulus of the steel is assumed from the literature.

5. The numerical model of the bridge

The FE model of the bridge assumed as a case study was developed by the software SAP2000 [34]. The various components of the structure were modelled considering their real geometry with mono-dimensional (frame) and bi-dimensional (shell) finite elements, obtaining a three-dimensional model including deck and piers restrained at the base by hinges. The concrete section was considered entirely reactive assuming that the prestressing effect was efficient.

The deck was modelled by thin “shell” elements, with 4 nodes and a formulation that combines membrane and plate-bending behaviour and the stress and strain are evaluated in the local coordinate system of the elements in 2x2 Gaussian integration points and are extrapolated in the connection nodes between the elements. The nodes of discretization of the deck are approximately 4600 with a mesh of 1 $\times$ 0.7 m for the upper slab.

The piers were modelled using frame elements for the 8 pillars and shell elements for the transverse elements at the top and the bottom. Moreover, tendon elements were added to the model to simulate the presence of prestressed tendons. Tendon elements have been inserted into the plane finite elements of the bridge model to simulate the effects of prestressing in the study of the bridge bearing capacity, that is not proposed in this paper, but in the structural identification their effect is negligible.

Links were used to connect the piers to the deck because the bridge was realized in two steps. First, the deck was simply supported on steel tubes located on the pears as cylinder hinges to apply the prestressing. Then, a concrete casting was realized around the steel tube to fully restrain the elements.

A rectangular beam was arranged along the two longitudinal sides of the bridge to consider the stiffness effect along the transverse direction due to the sidewalk added in 1982 by a steel structure bolted to the bridge.

The weight of the trucks was applied considering the position and the load of each axis and distributing the load of each wheel on an area of 40 cm $\times$ 40 cm, which is the conventional print provided by codes.

Fig. 6 shows the view of the model implemented by the software SAP2000 [34]. The elastic modulus of concrete was evaluated by the Eurocode 2 formulation [35] using the mean concrete cylinder strength measured by the in situ tests (46.1 MPa for the deck and 33.8 MPa for the piers) and resulted in 34700 MPa and 31700 MPa for the concrete of the deck and piers, respectively. Furthermore, a value of 196500 MPa from the literature was adopted for the steel tendons.

6. Model updating by the in-situ loading tests

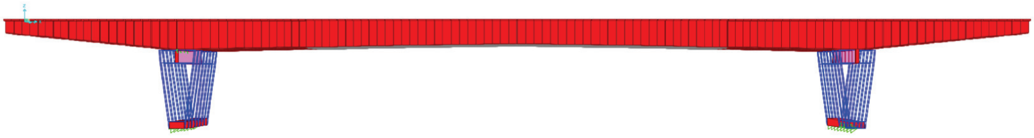
Usually, a measurement program based on static loading tests is aimed at identifying and checking the values of some important response parameters involving both material and structural properties. Here, the deflections from a static load test are used to update the preliminary FE model without giving comments on the performance of the bridge, which is not the aim of this study. The geometry of the structure is considered fixed and is excluded by the process of model updating because an accurate survey was developed, also in good agreement with the original design.

6.1. The in-situ loading test

The static load testing program has been designed and developed for attaining the serviceability stresses due to the traffic action NTC2018 in agreement with the Italian code [36], which is also the one of Eurocode [37]. The loads consisted of 44-ton trucks parked in different positions to produce five different loading conditions. The loading configuration is shown in Fig. 7 with a rectangle plus an arrow representing each truck and its orientation. In detail, in the first step (load case 1), 2 fully loaded trucks are placed on the cantilever span close to pier 1 and arranged in two lines. In configurations 2 to 4, the trucks are located at mid-span and arranged in two lines (with 2, 4 and 6 trucks, respectively). In the last load case 2, fully loaded trucks are placed on the cantilever span close to pier 2.

Displacements were recorded by utilizing total stations of the type Wild NA-2 in several locations (13 points for the north side and 13 points

a)



b)

Fig. 6. a) FEM model of the bridge including prestressed tendons; b) extruded view of the model.

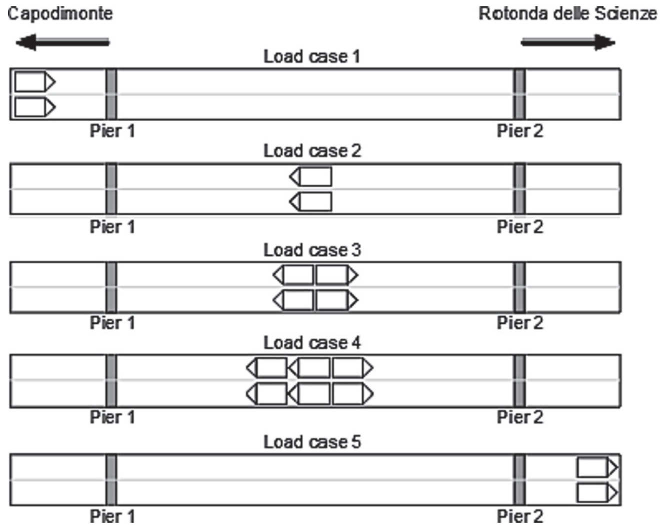


Fig. 7. Load configurations during the static load test.

for the south side), as detailed in Fig. 8, in the vertical direction to capture the deflection line. The measurement was taken within ten minutes for each loading case. All the equipment was calibrated before the measurements were taken.

By simply connecting the measured values of the displacement sensors, the deflection lines of the deck are drawn and shown in Fig. 9 for both the left and right sides (solid lines) for all the loading conditions.

A static FE analysis with the model described in section 4 has been performed to provide the numerical response prediction of the load testing. The weight of the trucks was applied considering the position and the load of each axis and distributing the load of each wheel on an area of 40 cm × 40 cm, which is the conventional print provided by codes. The results of the model are reported in Fig. 9 (dashed line) for comparison with the experimental results. The values of the displacements seem very small because the midspan deflection/span length is approximately 1/5000 because the bridge is very stiff for vertical loads due to the deck section and the hyperstatic scheme. However, it is necessary to consider that the design of the bridge was influenced by the high effect of the self-weight because the bending moment due to traffic is 40 % of the one due to the self-weight and by the necessary limitation of the displacement at the ends; in fact, the ratio between the theoretical displacement at the end of the cantilever and the cantilever length is approximately 1/1100. The smaller experimental values are discussed in the process of model updating.

The numerical deflection lines show good agreement with the experimental ones when the trucks are located at mid-span (load cases from 2 to 4), while they show evident differences when the trucks are located on the cantilevers (for load cases 1 and 5).

The results of the load cases applied on the cantilevers evidenced two boundary effects due to the actual situation of the bridge. The structure is not free at the extremes of the cantilevers because the experimental shape is completely different from the numerical shape due to the longitudinal displacement of the frame (actually, the dilatation joints are closed with cement, and the bridge is constrained between the two hills). Furthermore, the experimental vertical down displacements at the ends are lower than the numerical displacements, indicating a limitation under the structure. In fact, the gabions protect the hill side under and behind the bridge cantilever for the entire width of the bridge, but they were realized leaving a very small gap in the structure (Fig. 10), so that under the loads applied on the cantilever, the vertical displacement at the end of the deck is limited. This type of constraint is very difficult to introduce in the linear model because it is a unilateral limitation; however, the simulation by elastic restraint can allow calibration of the global response of the structure.

The results underline the importance of model updating to bring to light the real working conditions of the bridge and assess its conditions.

6.2. Model updating

The updating process of the FE model has been started and addressed according to the comparison between the experimental and numerical results of the initial FE model to reduce the differences. The procedure has been carried out analysing the influence of the structural behaviour on the measured parameters, which is an engineering approach. The target used to assess the model is the minimization of the factor $\Delta = (d_{exp}–d_{num})/d_{exp}$ considering the following significance measures:

- Vertical deflection at midspan d_m
- Vertical deflection at the end of cantilevers d_s and d_d
- The rotations of the deck at piers axes $d_{\phi s}$ and $d_{\phi d}$

Varying the structural parameters that influence the deformability response of the bridge, in particular, these structural parameters that are the boundary constraints and the elastic modulus as a linear behaviour of the structure is considered.

The updating process has been conducted in two steps. First, fixed boundary constraints have been introduced at the bridge ends in the longitudinal direction due to the type of response when the cantilever is loaded and the in-situ investigation, which clearly highlights the response of a longitudinal fixed frame. In fact, the gaps between the deck and the hillsides which are necessary to consider the structure as independent and free, have been filled with cement that offers a high stiffness in compression when longitudinal displacements are needed.

Then, the elastic modulus of the deck concrete and elastic boundary constraints at the ends in the vertical direction were calibrated. The actual conditions are truly more complex in the case of the vertical constraints at the end of the cantilevers because the displacement is not completely free due to the same cement filling and the contact with the gabions installed below the bridge as protection for the hillsides. Both these effects are not rigid and fully effective for restraining the vertical displacement; therefore, vertical elastic restraints, springs with stiffness F/L, linearly distributed along the width of the deck (stiffness/L) have been introduced in the model to be calibrated.

In Tables 1-3, the values of the control parameter Δ before and after updating are reported considering the measures that are more significant as a response for each load case: the end displacement, the rotation of the near support and the midspan displacement when the cantilever is loaded (load cases 1 and 5), and both the rotations and the midspan displacement when the span is loaded.

Different values have been calibrated for the restraint vertical stiffness at the two ends because the experimental response that appeared is not perfectly symmetric. The values of the restraint stiffness are 17,860

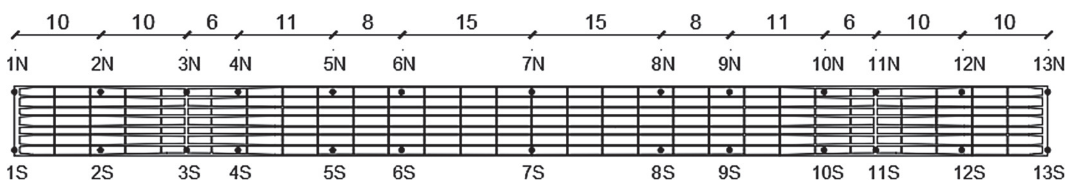


Fig. 8. Location of the points along the carriageway for displacement measurements.

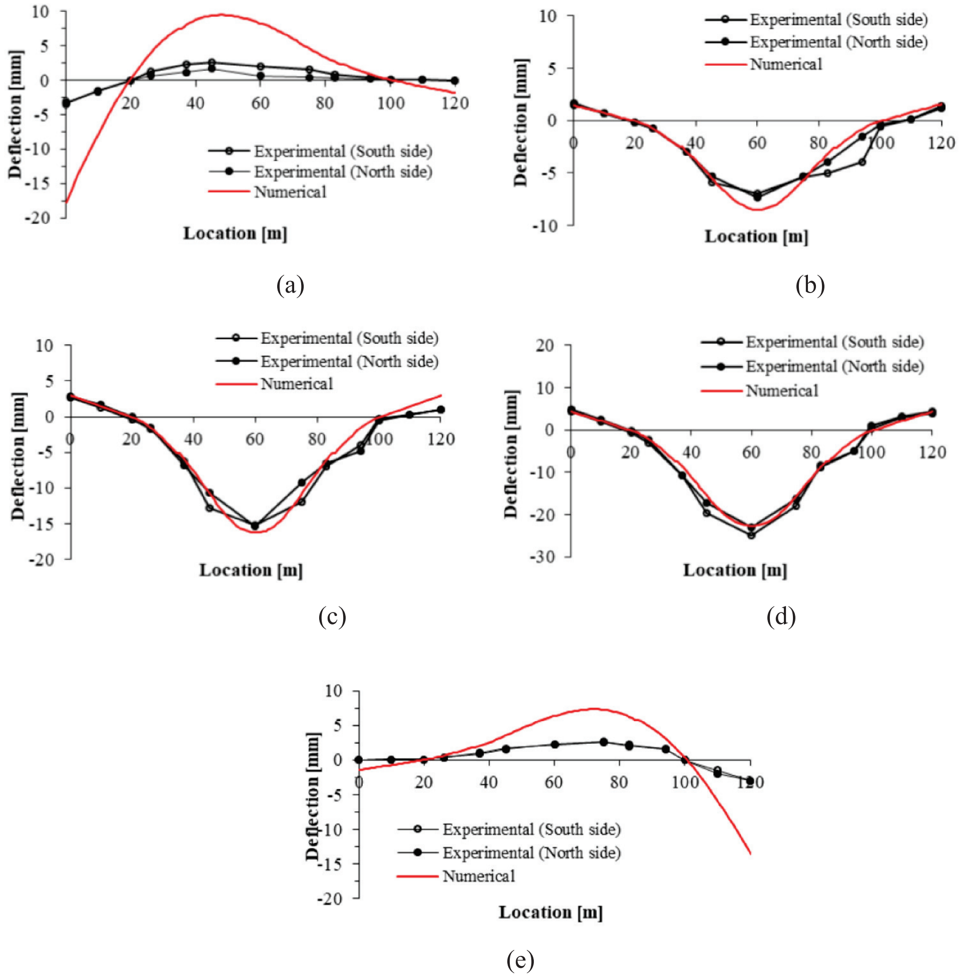


Fig. 9. Deflection lines of the deck under load cases 1 (a), 2(b), 3(c), 4(d) and 5(e).

Fig. 10. Details of the contact between deck and gabions.

Table 1
Values of the control parameters before and after updating for load case 1.

Parameter	Experimental	Beforeupdating		Afterupdating	
		Numerical	Δ [%]	Numerical	Δ [%]
d_m [mm]	1.31	8.27	532	1.77	35
d_s [mm]	−3.4	−17.71	421	−3.41	0.33
$d_{\phi s}$ [–]	0.00018	0.00066	272	0.00014	−23

Table 2
Values of the control parameters before and after updating for load case 5.

Parameter	Experimental	Beforeupdating		Afterupdating	
		Numerical	Δ [%]	Numerical	Δ [%]
d_m [mm]	2.27	6.38	182	1.57	−30.79
d_d [mm]	−3.01	−13.55	350	−3.02	0.43
$d_{\phi d}$ [–]	−0.00004	−0.00014	215	−0.00003	−21.86

Table 3
Values of the control parameters before and after updating for load case 4.

Parameter	Experimental	Before updating		After updating	
		Numerical	Δ [%]	Numerical	Δ [%]
d_m [mm]	−23.985	−22.68	−5.42	−23.71	−1.15
$d_{\phi s}$ [-]	−0.00033	−0.00022	−35.43	−0.00022	−34.18
$d_{\phi d}$ [-]	0.00075	0.00086	14.17	0.00090	19.59

kN/m^2 on the right and 21,620 kN/m^2 on the left, while the elastic modulus of the deck has a negligible variation (-5%) and results in 32965 MPa. The calibration of the vertical end restraints is governed by the response of the cantilevers in load cases 1 and 5, while the variation in the elastic modulus is due to the fitting of the response under the span loading of case 4.

The experimental–numerical comparison of the deflection lines applying the updated model is depicted in Fig. 12. Both the values of the control parameters and the graphical comparison indicate the reliability of the simple process and the large influence of the real constraints, while the elastic and geometrical characteristics of the bridge established by in situ investigation are substantially confirmed. The model updating has been addressed by the comprehension of the structure condition; therefore, the result is a confirmation of the model reliability and gives the information that are necessary to approach a rehabilitation design with a higher knowledge with respect to the visual investigation of these details. At the same time, it is important to understand the cause of the structural response giving a clear meaning to the numerical model because it is the only way to design a maintenance intervention.

7. Model updating by the in situ dynamic test

Dynamic tests by means of environmental vibrations are certainly an experimental technique less expensive than a load test because it is not necessary to place heavy vehicles in various configurations with a considerable saving of time and costs. However, the information that can be obtained on the behaviour of the structure is very useful for validating a numerical model, only if the postprocessing of the data is correctly conducted both for the elaboration of the measures and the model updating.

The dynamic test cannot provide information on the response of the structure and its capacity in strength and ductility in the nonlinear field. The dynamic tests carried out on the case study are briefly described in the following section as a starting point for the model updating process. The first results of the updating process are reported in [38], but the results reported herein have been obtained with further accurate analysis and review of the model.

7.1. The in situ dynamic test

The ambient vibration test (AVT) was chosen as the dynamic test because it does not require any additive equipment and affects the use of the bridge for a short time; therefore, it is also a perspective of monitoring by periodic execution.

The dynamic behaviour of the bridge was analysed by means of the previously introduced numerical model to establish the layout of the sensors with 27 measuring points, as shown in Fig. 13. To identify both the bending, in plane and out-of-plane, and torsional modes, sensors were arranged in pairs at the long sides of the roadway with a distance of 6 m along the cantilever and 10–12 m along the central span.

The adopted instrumentation consists of a 16-channel data acquisition system with 5 triaxial MEMS piezoelectric accelerometers, characterized by an integrated Sigma Delta 24-bit A/D converter, with a nominal sensitivity of approximately 1 V/g and a fixed sampling frequency of 1024 Hz; the accelerometers have been installed by means of special metal bases fixed directly on the deck with expansion anchors and connected to a centralized data acquisition board by means of long transducer cables.

Since only 5 triaxial accelerometers were available, more test setups were performed and then combined to cover the 27 measurement points of Fig. 11. These setups shared a sensor in common (placed at point 6 in Fig. 11), the so-called reference sensor that was fixed for all the measurements, while the other sensors were moved from one setup to the next. Therefore, to obtain the modal parameters of the investigated bridge, it was necessary to process the data of all the measurement setups and normalize it as the unmeasured background excitation of each setup might be different [39,40].

In each setup, the accelerations were acquired with environmental conditions only (wind, passage of persons) [41] for a duration of 30 min at a frequency of 1024 Hz.

The environmental records carried out at the measurement points have been inspected and validated. Subsequently, each recording was filtered by the application of a Butterworth-type bandwidth filter of order 6 in the range 0.2–20 Hz. Subsequently, the time series were decimated to obtain a final sampling frequency of 20 Hz. It is worth noting that filtering performed before decimation ensures the absence of aliasing problems in the frequency range of interest.

The data processing and the extraction of modal shapes were carried out by the software ARTeMIS [42]. For the cases of multisetup data, i.e., when several test setups have been measured using a set of fixed positioned reference sensors and a set of rowing sensors being moved across a structure from measurement to measurement, the nonsynchronous acquisitions should be combined. Generally, this problem can be addressed with two approaches [43,44]: the postidentification method that processes all the setups separately and merges them at the end and the preidentification method that processes all the setups together. In this latter case, the different measurement setups can be processed in one step and do not have to be analysed separately.

In this paper, the preidentification method described in [45,46] has been adopted. More specifically, the SSI-UPC Merged Test Setups algorithm available in ARTeMIS Modal Pro [43] based on the data driven

(a) (b)

Fig. 11. Details of the gap between the deck and the hillsides.

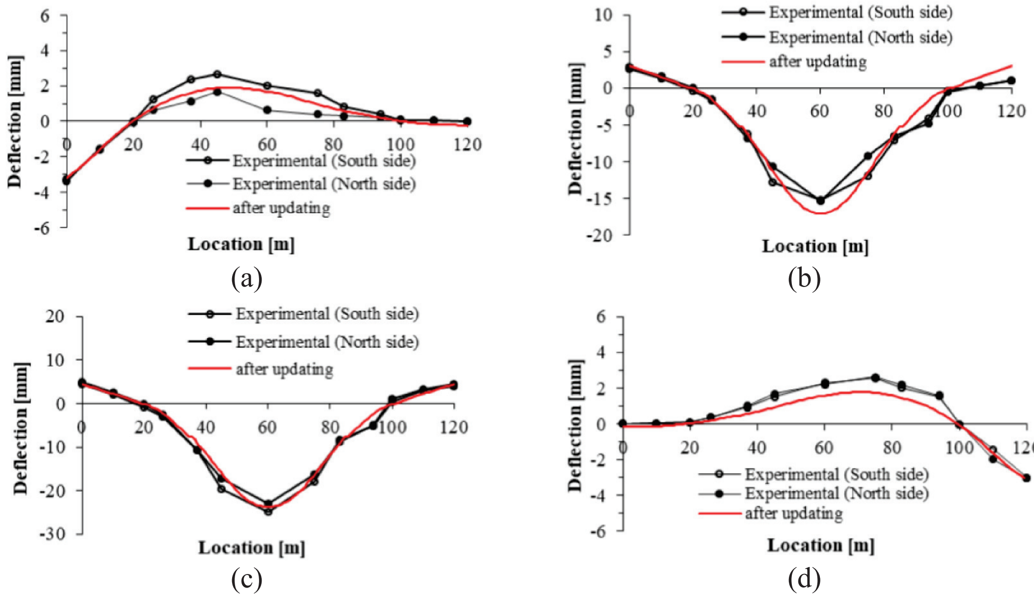


Fig. 12. Deflection lines of the deck after updating under load cases 1 (a), 2(b), 4(c) and 5(d).

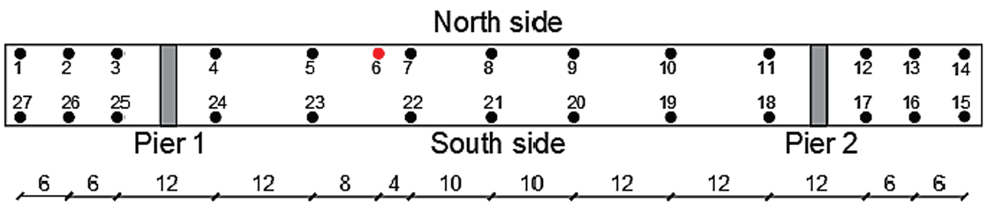


Fig. 13. The layout of sensors in the dynamic test.

Stocastic Sub space Identification (SSI) method [47] has been used. The algorithm can be briefly described by the following steps: (i) multiple time series measurements called "Test Setups" are introduced as input to the algorithm; (ii) these measurements are compressed into a set of so-called Common SSI matrices, which are then merged together using a scaling technique that utilizes the information of the references sensors of the Test Setups. This scaling technique is introduced to account for a potential change in the vibration level from measurement to measurement. (iii) The estimation of modes is then performed using a single stabilization diagram that displays the modal parameters of the range of state space models being estimated. The estimation of the state space models is made using the fast implementation of the Crystal Clear SSI estimator.

In this case, the common SSI matrices were estimated using a Max. Dimension of 100. The Crystal Clear SSI estimator was applied using a Max. No. of Eigenvalues setting of 38 that was automatically chosen by the algorithms. The global modes were automatically estimated based on the stable modes of the diagram reported in Fig. 14. The criterion adopted for stable modes was a maximum deviation of 0.0001 Hz of natural frequencies, 10 % damping ratios and 5 % MAC values between a mode for increasing system order.

Three global modes were selected and considered for the subsequent updating process. The modal shapes and values of the natural frequencies for each of the three selected experimental modes are reported in Fig. 15. The first mode in Fig. 15(a) is a horizontal transverse bending mode of the whole bridge with a frequency of 1.19 Hz. The lateral fluctuation reaches the largest value at the bridge centre, and the shape is symmetric. The second mode in Fig. 15 (b) is dominated by the vertical fluctuation of the main span of the structure with a frequency of 2.12 Hz, and the shape is symmetric with the maximum displacement at the middle span. The third mode in Fig. 15 (c) is vertical-torsional with a frequency of 5.64 Hz. The two longitudinal sides of the bridge fluctuate along the two opposite vertical directions. In this case, the experimental shapes are not perfectly symmetric with respect to the bridge centre because the third mode is more affected by imperfections of the bridge and measures accuracy.

In addition, the identified modes are all normal modes, as seen from the examination of the complexity plots shown in Fig. 16. It is hardly necessary to point out that the imaginary components present in some modes are minimal, and clearly attributable to the effects of the measuring noise.

The initial numerical model provides, among the first nine mode shapes, the experimental mode shapes identified with the dynamic test. In particular, the experimental modes correspond to the third, seventh and ninth numerical modes. In Fig. 17a, the transverse displacements (along Y) are depicted, while in Fig. 17b and Fig. 17c, the vertical displacements (along Z) are presented.

It is worth noting that the numerical vertical mode shapes are in good agreement with the experimental shapes, while the transverse and torsional modes do not match the experimental shapes. The latter suggests that in the numerical model, the deck at the cantilevers should be restrained along the transverse direction and in the vertical direction but also in the longitudinal direction because the longitudinal modal shape was not measured.

The consistency of the comparison was also measured in terms of frequency error ε and MAC values [48]. The results are reported in Table 4.

As can be noted from Table 4, the assessment between numerical and experimental returns a good agreement in terms of frequency for transverse and vertical modes, while a greater error (15 %) is detected

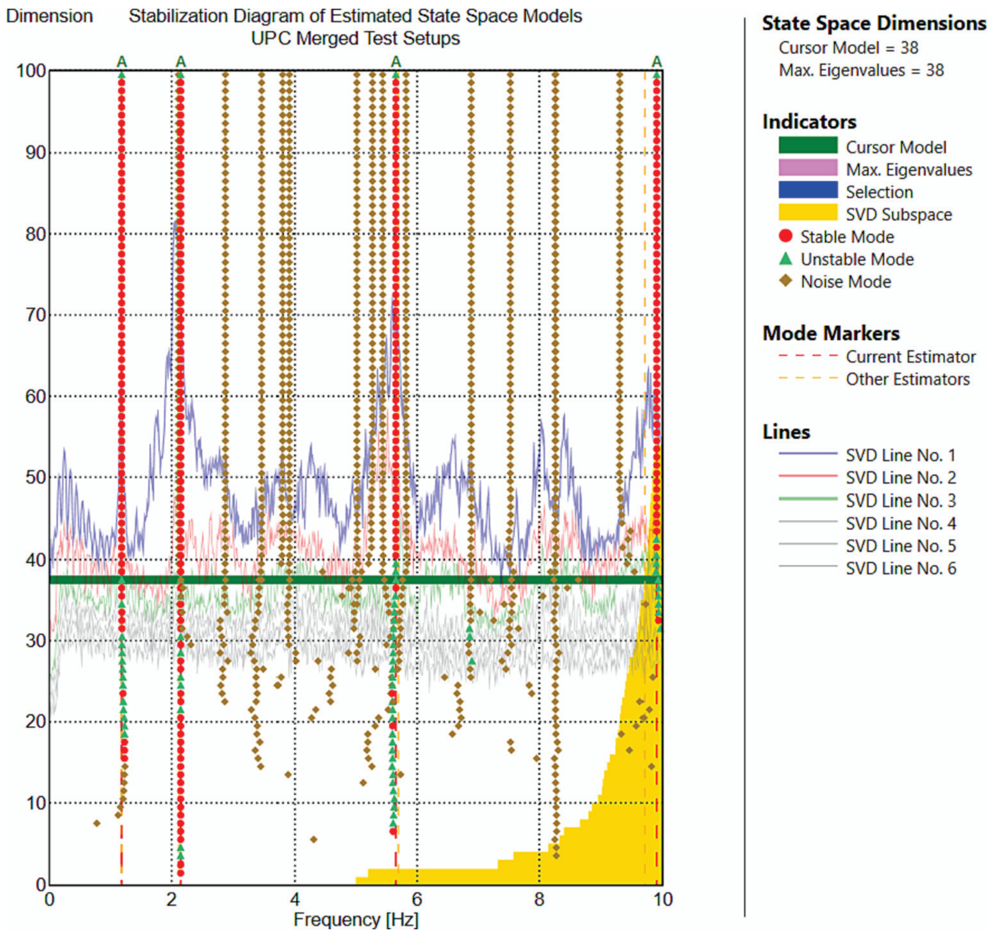


Fig. 14. Stabilization diagram of the case study.

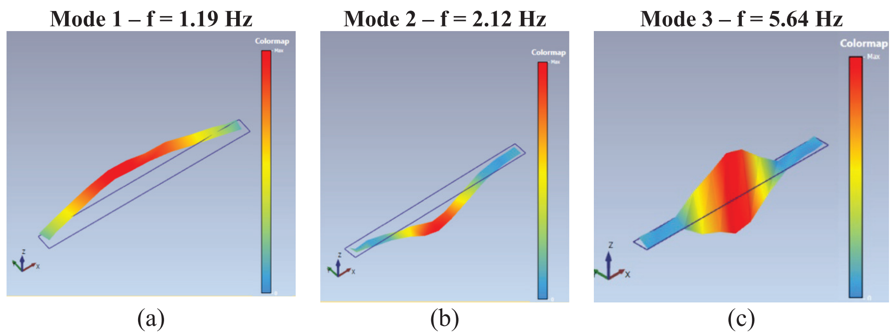


Fig. 15. Experimental mode shapes: (a) lateral bending mode; (b) vertical bending mode; (c) vertical torsional mode.

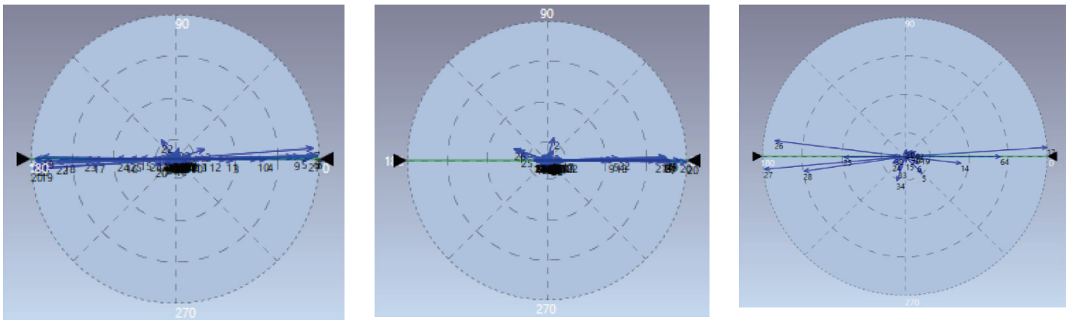
for the torsional mode of the deck; furthermore, the numbering of the modes isn't consistent between the numerical and experimental returns.

This first comparison between the experimental and numerical results in terms of dynamic behaviour already confirms that the bridge is constrained along the longitudinal and vertical directions at the end; therefore, in this case, it is also necessary to update the model to identify the real conditions of the structure.

7.2. Model updating

As in the case of the load test, the model needs to be calibrated, especially in terms of constraints at the edges that avoid or reduce the movement of the bridge. Therefore, a first manual updating of the model was performed by changing the following prominent items in agreement with physical phenomena due to real conditions observed during the investigation, as already explained in the case of the load test. In particular, the following changes were applied to the model in the first step:

- fixed supports in the transverse and longitudinal directions have been inserted at the ends of the cantilevered spans to take into account the longitudinal and transverse displacements being restrained because the gap between the bridge and the road is filled with cement;
- elastic supports (springs with stiffness F/L), linearly distributed along the width of the deck (stiffness/L), have been applied at the ends in the vertical direction to take into account the gap below the



Mode 1 Mode 2 Mode 3

Fig. 16. Complexity plots of the experimental mode shapes.

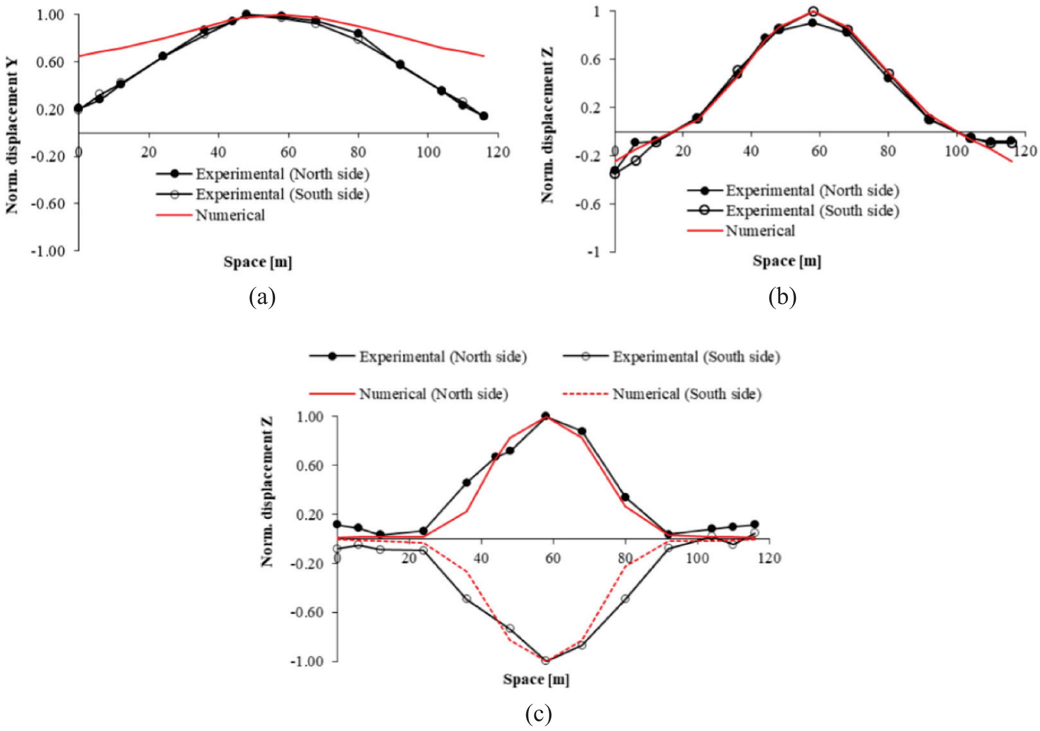


Fig. 17. Comparison between experimental and numerical mode shapes: (a) transverse mode; (b) vertical mode; (c) torsional mode of the deck.

Table 4
Theoretical-experimental comparison.

Experimental		Numerical			
Mode N.	f_{exp} [Hz]	Mode N.	f_{FEM} [Hz]	ε [%]	MAC
1	1.19	3	1.20	0.47	0.80
2	2.12	7	2.11	−0.38	0.96
3	5.64	9	4.78	−15.25	0.97

cantilevers of the bridge not being sufficient to avoid the constraint effect of the gabions and reinforced soil.

The results give a better correspondence between the order of the modes and the MAC of the higher modes, but the increment of the bridge stiffness due to the added constraints increased the frequency of the first mode, which was in good agreement with the initial model; therefore, further variation was considered for reducing the transverse bridge stiffness that directly influences the frequency of the first mode:

- horizontal transverse restraints at the ends are considered not fixed but elastic (springs with stiffness measured in Force/Length), linearly distributed along the width of the deck (stiffness/Length), because the effect of the gap filling is not completely effective;
- the elastic modulus of the concrete for the piers was reduced to consider the degradation phenomena of the concrete cover evidenced during the in situ survey;
- the elastic modulus of the concrete for the deck was reduced a little.

All the above considerations, properly introduced in the numerical

Table 5
Theoretical-experimental comparison after updating.

Experimental		Numerical	Comparison		
Mode N.	f_{exp} [Hz]	Mode N.	f_{FEM} [Hz]	ε [%]	MAC
1	1.19	1	1.36	14.7	0.98
2	2.12	2	2.14	1.0	0.97
3	5.64	3	4.90	−13.1	0.98

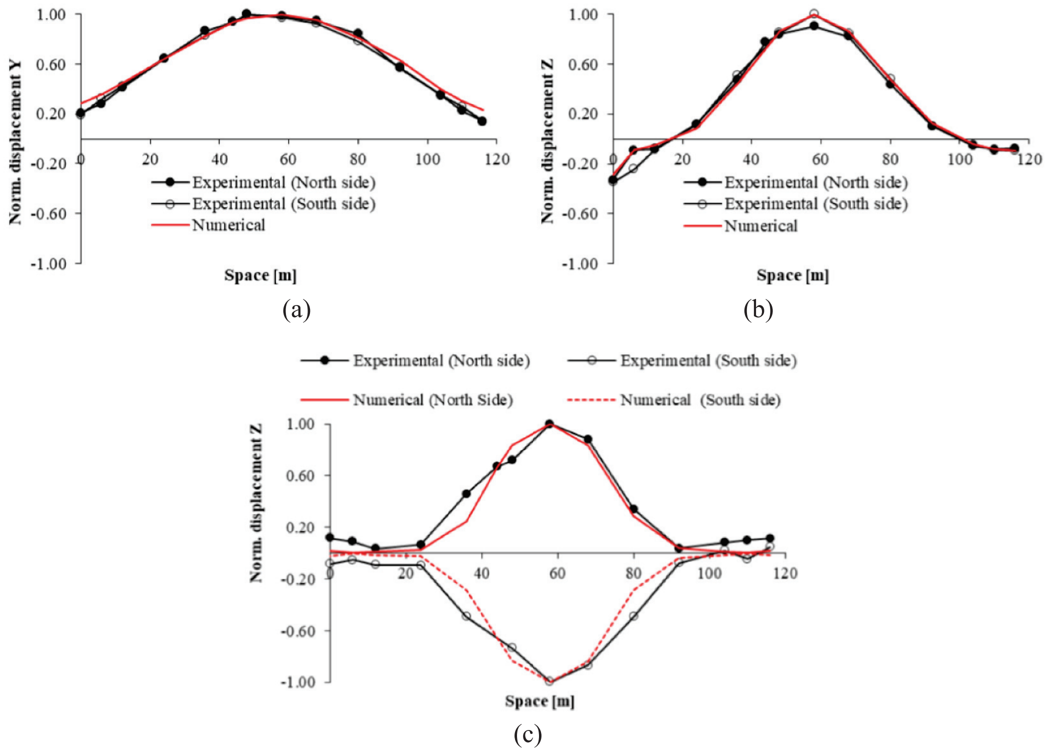


Fig. 18. Comparison between experimental and numerical mode shapes after dynamic updating: (a) transverse mode; (b) vertical mode; (c) torsional mode of the deck.

model, finally modified the mode shapes of the model and led to the correspondence of the order between experimental and numerical modes. In Table 5 and Fig. 18, the comparison of numerical and experimental modes after updating is shown. An improvement of the correspondence between numerical and experimental behaviour is also confirmed by the increment of MAC, confirming the good addressing of the procedure.

The values of the restraint stiffness are 21,620 kN/m^2 on the right and 17,860 kN/m^2 on the left in the vertical direction, while the elastic modulus has a negligible variation for the deck (−5%) and a greater variation (−20 %) for the piers. Furthermore, elastic restraints with a stiffness of 33,000 kN/m^2 on the right and 28,000 kN/m^2 on the left have been calibrated in the horizontal transverse direction at the edges.

The dynamic test allows us to calibrate the elastic modulus of the piers, which has an important role in the transverse stiffness of the structures excited by the lateral vibration, while in the load test, the longitudinal stiffness comes in the response as a rotation restraint of the deck with a high moment of inertia of the piers, making it difficult to calibrate the elastic modulus. Essentially, the numerical frequency of the first mode is still higher than the experimental frequency of 15 %, but a further reduction in the elastic moduli increases the error of the higher modes and is not compatible with the in-situ investigation of the material.

However, a reduction in the transverse horizontal stiffness, which only modifies the dynamic response of the frequency of the first mode, can be attained by introducing a deformability at the base of the piers due to soil-structure-interaction (SSI), as demonstrated by other studies [49–51]. This topic is not within the scope of this paper and requires an accurate study because it could give a considerable effect [52]; however, a possible effect of SSI can simply be considered by introducing an elastic spring at the base. The stiffness of the spring was calibrated by fitting the experimental frequency of the first mode until the error was reduced to approximately 5 %, that is, for a stiffness of 1000 kN/m (Table 6).

Table 6
Theoretical-experimental comparison after updating introducing SSI.

Experimental	Numerical	Comparison			
Mode N.	f_{exp} [Hz]	Mode N.	f_{FEM} [Hz]	ε [%]	MAC
1	1.19	1	1.26	5.7	0.96
2	2.12	2	2.14	1.0	0.97
3	5.64	3	4.90	−13.1	0.97

Clearly, the error can be annulled but in this case, it cannot be supported by an engineering analysis because in this work the model of the foundation and soil have been excluded, even while the study is in progress.

8. Discussion of the results

The calibration of the numerical model has been implemented by two separate processes using the results of the load and dynamic tests to better understand the efficiency of each type of in situ test and their peculiarity; thus, the results are compared in Table 7 and discussed in the following section.

It is clear that some results can be assessed by both the tests obtaining the same identification as it occurs for the longitudinal fixed restraint and the elastic behaviour of the vertical constraint. Conversely, the transverse constraint and stiffness can be excited only by the dynamic oscillation and measured through the accelerometers because this effect by the application of vertical loads is more difficult to induce and measure. Therefore, the reduction in the elastic modulus of the piers obtained by the dynamic test with respect to the results of the load test cannot be interpreted as a difference between the static and dynamic elastic modulus but is due to the different efficiencies of the two tests in allowing its calibration. Additionally, the effect of SSI as the deformability of the base restraint of the piers can be evidenced by the dynamic test in the transverse vibration but is less detectable by the static

Table 7
Results of the static and dynamic model updating.

	Left edge			Right side			Elastic modulus (MPa)	
	Vertical stiffness (kN/m^2)	transverse constraint (kN/m^2)	longitudinal constraint	Vertical stiffness (kN/m^2)	transverse restraint (kN/m^2)	Longitudinal constraint	pier	deck
Load test	21,620	free	fixed	17,860	free	fixed	32,700	32,965
Dynamic test	21,620	33,000	fixed	17,860	28,000	fixed	26,160	32,965

response under vertical loads.

Furthermore, the load test does not confirm the dead load considered in the model, while the dynamic response depends on the masses, and the updating process confirms the good performance of the model. The contribution of the dynamic test in the assessment of the model is confirmed if its updating process is applied to the model already calibrated by the load test.

In Table 8, the dynamic characteristics of the model reported in par. -6.2 after updating by the load test are compared with the experimental results. The overlapping mode shapes are reported in Fig. 19.

The dynamic response of the model calibrated by the load test is in better agreement with the experimental results, especially for the torsional and vertical modes, but the transverse behaviour is clearly different, and the sequence of the modes is different. In fact, this last aspect is not negligible because it could influence the seismic response of the bridge in the transverse direction, which probably gives the worst performance of the structure under the action of an earthquake.

After this discussion the role and the complementarity of the two types of tests in the assessment of the model is clear, but another note is important. The load test can provide information on the type of response, linear or nonlinear, under serviceability traffic, which is a very important characteristic of bridge safety. Conversely, the dynamic test could demonstrate damage/degradation of the structure if its effect modifies the dynamic modal response, which means a not negligible modification of the constraints or the stiffness without the effect of a load.

9. Conclusions

The study proposes a general focus on the efficiency and complementarity of load and dynamic tests in the structural identification of bridges, which are usually applied separately and with different aims. The complete procedure of the model assessment, i.e., a structural identification is applied to an existing bridge considering two separate methods based on the in-situ load and dynamic tests and then discussing the different contributions to the knowledge of the structure. The selected case study is a bridge designed by Riccardo Morandi in the 1950s that is particularly complex for the static scheme, the deck and pier section but also for the real boundary conditions with respect to the original design.

The analysis and the discussion of the results provide general provisions for the use of the experimental tests in the process of model assessment using an engineering approach with supervision of the calibration process, evidencing these general features:

Table 8
Theoretical-experimental comparison after static updating.

Experimental		Numerical			
Mode N.	f_{exp} [Hz]	Mode N.	f_{FEM} [Hz]	ε [%]	MAC
1	1.19	1	0.86	−27.5	0.73
2	2.12	3	2.19	3.1	0.97
3	5.64	5	4.81	−14.8	0.83

- The dynamic AVT is a suitable procedure to define the behaviour of a structure in a linear field because it does not require time and organization for load application; however, a reliable numerical elaboration of the results is necessary with more than one technique. It is efficient to define the real constraints in the horizontal and vertical directions, even if friction could affect the role of the restraints because low stress is produced by ambient vibration. The dynamic test also allows us to assess the dead load of the model because the masses have a role in the response of the structure. The model calibrated by this type of in situ test can provide important reliability for seismic analysis, in which the dynamic response of the horizontal plane is also a key issue. In fact, the damage condition of the bridge can be detected only if its extension and type modify the global or local stiffness without the effect of loads, for example, opened cracks under dead loads or the diffuse cracking of the concrete cover, as for the piers of the case study;
- The model calibration by load tests does not confirm the dead load considered in the model but gives clear information of the linear or nonlinear response (damage detection) under serviceability traffic. The vertical constraints and the vertical stiffness can be identified, while in general, the transverse stiffness and the horizontal constraints cannot be defined. In the case of the study, the static pattern of the frame with two cantilevers allowed us to understand the effect of the longitudinal constraints, but in simple supported decks, this is usually not possible.

The process of the model assessment by an engineering supervised process also gives important specific results to address some interventions on the bridge.

The gaps at the end of the cantilever are filled and do not allow the structural response considered in the original design to change, especially the stress in the piers. Most likely, the particular situation increases the restraint level of the structure, reducing the stresses, but dangerous actions are applied on the unstable hills; furthermore, the different behaviour of the bridge with respect to the theoretical behaviour based on the original design could make an upgrade intervention inefficient.

The static and dynamic tests confirm the linear behaviour of the bridge and is in good agreement with the model in terms of dead load (masses) and stiffness (frequency and displacement under loads).

The complementarity of the dynamic and load tests has been evidenced, indicating that a robust assessment of an existing bridge needs both tests; however, for the model assessment in the elastic field, the dynamic AVT is the better solution if a linear analysis in the seismic field has to be developed, while the load test does not allow us to assess the horizontal response of the structure and the masses. The importance of an engineering approach is clearly evidenced by the discussion of the results which, through the assessment of the model, gives specific information on the structure only if the numerical response is related to physical situations observed by an accurate survey.

CRediT authorship contribution statement

Alessandra De Angelis: Methodology, Data curation, Software, Visualization, Investigation, Validation, Writing – original draft, Writing

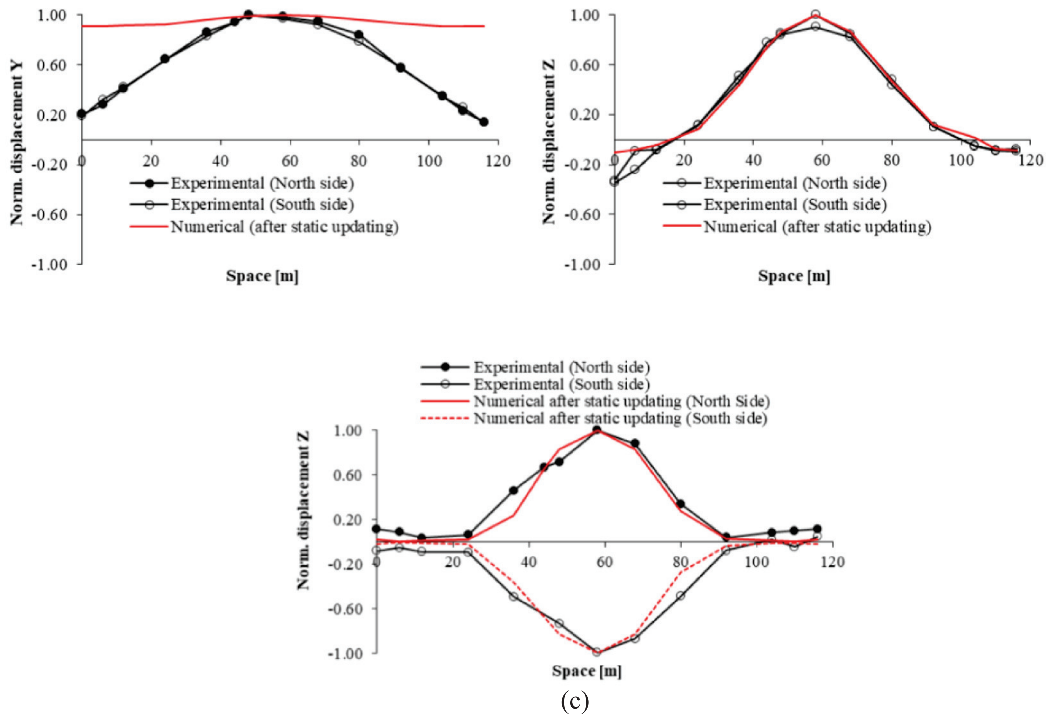


Fig. 19. Comparison between experimental and numerical mode shapes: (a) transverse mode; (b) vertical mode; (c) torsional mode of the deck after static updating.

– review & editing. **Maria Rosaria Pecce:** Conceptualization, Supervision, Writing – review & editing.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

No data was used for the research described in the article.

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