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Dynamic monitoring of bridges during static load tests: Influence of the dynamics of trucks on the modal parameters of the bridge

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Abstract

This paper presents results of an extensive experimental campaign carried out during the construction phases of a base-isolated bridge across the Potenza river in central Italy. The structure is constituted by a half-through steel arch bridge composed by two coupled weathering steel tubular arches sustained by thirty couples of hangers and by two approaching viaducts. The bridge has a continuous twin-girder steel-concrete composite deck. During construction, impact load tests were carried out on hangers as well as on other tie elements to estimate their tension force from the relevant resonance frequencies. During the proof test, relative displacements occurred between the upper and lower plates of the seismic isolators are measured for different loading conditions. In addition, ambient vibration tests are performed during the final static load tests to investigate the bridge dynamics subjected to different loading conditions. Comparisons of test results with the numerical predicted values contributed to evaluate the consistency between the design and the real structure at different construction stages. Dynamic identification tests contributed to identify the significance of the truck-bridge interaction in the interpretation of the bridge dynamics evaluated experimentally.

Keywords: bridge dynamic testing; modal identification; operational modal analysis; steel arch bridge; truck-structure interaction.

1. Introduction

A good performance of very complex engineering systems depends not only on an accurate design but also on their correct realization during all construction phases. As an example, strain and stress state of steel-concrete composite, suspension or cable-stayed bridges at the end of construction (unloaded condition) may strongly depend on the construction phases and may affect the bridge behaviour at service conditions. Thus, the structural behaviour must be coherent with the expected one resulting from design numerical models at all construction phases; in this sense, testing the

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structure during the construction may be important as well the final test, which is used to assess the behaviour of the bridge in its as-built condition with no previous loading or excitation (e.g. traffic loads, seismic action) [1]. Other examples of structural systems for which tests during the construction phases as well as at the final state may be important are bridges with special devices, such as footbridges with TMDs (Tuned Mass Dampers) for the vibration control for which a fine-tuning based on the actual modal properties of the completed footbridge should be done before they are installed [2].

Modern technical codes generally allow a design by testing and suggest test typologies that can be carried out with different aims, for instance to improve and update the design model consistently with results of mechanical tests on materials or structural elements (e.g. ultimate resistance, elastic properties) (e.g. [3-5]). With reference to tests performed at the end of the structure construction, aimed to enable the construction for use, technical codes have always prescribed for bridges a mandatory final proof load test, to be performed before the opening to traffic. The latter consists in a static test usually carried out by loading the bridge with fully loaded trucks and in the evaluation of the maximum vertical displacement of the span, which is compared to the numerical predicted value through the design model, and its residual value after unloading. Furthermore, some technical codes have included in the final testing procedure also a dynamic load test that usually consists in evaluating the modal properties of the unloaded bridge, e.g. resonance frequency values and the relevant mode shapes and damping ratios, which must be compared to those numerically evaluated [6-9]. Italian technical code released in 2008 [10] only suggested the execution of dynamic load test whereas the updated version of 2018 [11-12] considers this test as mandatory for bridges of significant importance. However, the mandatory check is limited to the first vibration frequency that must be comparable with the numerical prediction. Despite codes seems to address a minor importance to dynamic tests, many recent research papers are focusing on issues such that the dynamic identification or the structural health monitoring of bridges [13-19].

Generally, in case of complex structures (such as the steel arch bridge considered in this study), several checking and tests are performed during construction stages and more exhaustive static and dynamic tests are carried out at the final testing [20-21]. About the latter, a more complete dynamic characterization including the identification of several vibration modes is often required with the aim to catch the 3D flexural-torsional behaviour of the bridge, as well as the transverse one. In addition to traditional forced vibration tests carried out with a fully loaded truck passing over a bump positioned on the road (inducing a quasi-impact load mainly exciting the flexural-torsional modes) or sharply accelerating along the road (to mainly excite the longitudinal modes), Ambient Vibration Tests (AVTs) are more recently considered as one of the most effective method, fast and cheap, to

check the global dynamic behaviour of a bridge. Such technique makes also simple obtaining the dynamic characterization of the bridge during the different phases of the final static test (in the unloaded, loaded and again unloaded configurations), by leaving the sensors in the same position on the bridge during the final static test. Evaluating the modal properties of the bridge in the loaded configuration in addition to the traditional unloaded configuration can add useful information on the consistency between the real (experimental) and expected (numerical) behaviours of the bridge. Trucks used for the bridge loading are usually considered as added masses on the numerical model of the bridge. Indeed, they are complex dynamic systems and the effects of their dynamic interaction with the bridge can be not negligible, especially for those modes characterized by frequency values enough close to those of the truck. In these cases, the frequency values measured on the bridge deck could be significantly different than that expected by modelling the trucks as just added masses. This may generate possible misunderstandings in the interpretation of the experimental frequency that can lead, in some cases, to a negative judgement of the final dynamic testing.

In the literature there are several studies on the dynamic interaction between movable vehicles and bridges mainly devoted to investigate the changes in the bridge dynamic behaviour (e.g. dynamic amplification, modal parameter changes) induced by vehicles travelling on the road, depending on the road and vehicles characteristics, e.g. road roughness and unevenness and vehicles velocity [22-25]. More recently researchers have also studied how the vehicle bridge interaction may affect the dynamic response of the bridge during earthquakes [26-27].

However, very few studies are available in the literature, dealing specifically with the interaction problem between stationary vehicles and bridges and addressing how their presence on the deck can modify the dynamic properties of the bridge, such as the natural frequencies and mode shapes.

This paper presents a program of combined static and dynamic tests performed on a base-isolated steel-concrete composite half-through arch bridge at different construction phases, including the final configuration. The bridge was recently built across the Potenza river, on the No. 77 main road in Villa Potenza, close to the town of Macerata in central Italy and is composed by a main span (arch) of about 115 m and by one approaching viaduct at each side, constituted by continuous steel-concrete composite decks connected to that of the arch span. The experimental program aimed to get useful information during the bridge construction (e.g. forces on hangers, seismic isolator displacements and global dynamic properties) necessary for excluding construction anomalies and for calibrating a reliable numerical model for the interpretation of the final proof test. Dynamic tests are performed at different loading conditions (unloaded and loaded bridge) and suitable numerical models are developed to interpret the relevant results, with a special attention to the truck-bridge interaction.

2 The bridge on the Potenza River in Villa Potenza

The bridge on the Potenza River along the main road n. 77 in Villa Potenza, in central Italy, is composed by a half-through arch bridge with a span of about 115 m and by one approaching viaduct at each side, for a total length of 285 m (Fig. 1a). The arches (Fig. 1b) are constituted by two coupled weathering steel hollow elements with a rhomboidal cross-section, which are inclined with respect to the vertical plane and connected just above the foundation by two transverse beams to eliminate the horizontal thrusts, and near the top by two short transverse beams, to minimize the arches stability problems. The rhomboidal cross-section is suitably oriented to assure the greatest stiffness and strength of the arches with respect to transverse actions (out of the arch plane) (Fig. 1c).

The bridge is founded on piles (pile of diameter between 1.0 ÷ 1.2 m are used). With respect to the single arch solution, the double arch allowed the reduction of the overall dimensions of foundations in order to minimize the interaction between bridge substructures and river flow. The arches sustain a steel 8.9 m spaced twin I-girder directly supporting cross-beam with cantilevers deck by means of inclined pre-tensioned steel hangers. The steel I-shaped cross-beams, with spacing 4.50 ÷ 5.00 m and variable height, support a reinforced concrete slab with a medium thickness of about 0.27 m. Fifteen pairs of hangers are positioned on the upstream and downstream side of the deck, respectively, with a total of 60 elements. Hangers, having a uniform circular cross-section with diameter of 64 mm and variable length (3.59 m to 13.79 m), are equipped at both ends with a steel fork shaped monolithic element through which a spherical joint and a cylindrical pin constrain are realized at the connection with the arches and with the deck, respectively. The turnbuckle, for the hanger pre-tensioning, is located about 1.5 m high from the deck.

Depending on the loading configurations, tensile or compressive forces arise at both ends of the central arch span (Fig. 1a); lifting forces are entrusted by means of four groups of eight vertical steel ties (hereafter called deck-end ties), made of the same steel elements adopted for hangers, which connect the deck with a specific foundation. Compression forces are entrusted to hollow circular steel members which are connected to the deck through elastomeric bearings (Fig. 1g). Ties at the North-East side are 2.4 m long, while those placed at the South-West side are 4.65 m long.

Each approaching viaduct is 85 m long and consists of a three spans steel-concrete composite deck supported by two intermediate piers and an abutment, both founded on piles (Fig. 1e). On one side of the deck a slightly curved pedestrian walkway is built (Fig. 1f) adjacent to the roadway at the bridge ends and with maximum distance from that of about 1.50 m at midspan. According to designers, this layout is conceived with the aim of giving to pedestrians the perception of walking not along a roadway but on a walkway with a more natural aspect. This perception is also increased by the fact that the hangers of this side of the bridge are differently inclined and are not in the same plane

of the arch: some of them are external to the walkway, whereas those at the midspan are internal.

The bridge is located in a seismic prone area with medium-high hazard intensity. In order to avoid damage and preserve functionality in the case of strong earthquakes, the whole structure system is protected by a “partial isolation system” [28-30]. This solution is quite advantageous for decks with a high ratio between total length and transverse width because it permits both to strongly reduce the seismic actions and to avoid transverse displacements at abutments [31-32].

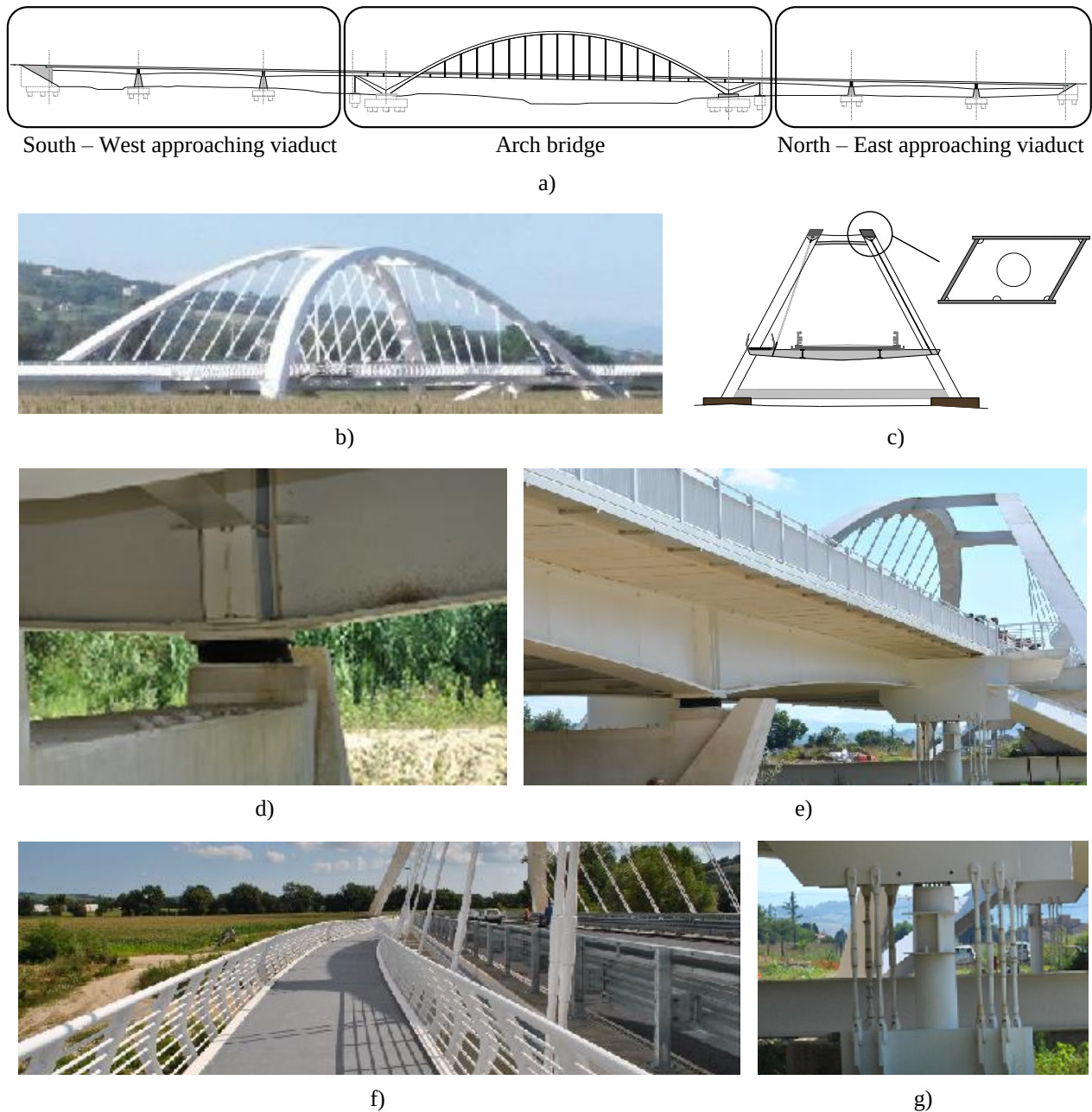


Figure 1. The bridge on the Potenza river: a) global scheme, b) main arch bridge, c) arch cross sections, d) rubber seismic isolators, e) approaching viaduct, f) pedestrian walkway, g) deck-end ties and hollow circular steel members.

The seismic isolation technique is based on the stiffness of the superstructure, this result has been achieved by the arch-tie solution adopted in the longitudinal direction (Fig. 1a) and by the trapezoidal scheme with bottom tie in the transverse direction (Fig. 1c). High Damping Rubber Bearings (HDRBs) (Fig. 1d) with different properties have been used to accomplish the different displacements and axial forces expected at the bearings of the approaching spans and the central arch.

Steel connections of all the bridge are welded to improve durability, to make easier the inspection and to increase the aesthetical quality. In addition, for similar reasons, steel elements are painted with four coats of white paint.

3 Experimental program

The experimental campaign is planned with the aim to monitor some specific structural components as well as the overall bridge response at different stages during the construction process and during the static load test required for the bridge opening. In particular, the tensile forces in hangers and deck-end ties, the relative displacements of the elastomeric bearings and the overall dynamics of the bridge are controlled during the construction phases and/or the final state (Fig. 2). Construction phases are: the end of the steel work construction (P1); the beginning of the concrete slab casting (i.e. with only predalles and slab reinforcements positioned on the deck) (P2); the end of the bridge construction (i.e. with also non-structural dead loads) (P3) and finally the configuration corresponding to the static load test (P4). With reference to P4, experimental tests are performed at three different loading conditions (Fig. 3): configuration U refers to the Unloaded condition while C1 and C2 refer to the 2 loaded Configurations of Fig.3. In detail, in configuration C1, 10 fully-loaded trucks (about 36 t per truck) are located at mid-span and arranged in two lines (the first with six trucks and the second with four) while in configuration C2, 5 fully-loaded trucks are placed close to a bridge support (arranged in two lines of three and two trucks, respectively). Tests are aimed to identify and/or check values of some important response parameters involving both material and structural properties. With reference to hangers, frequency values are identified during all construction phases (P1÷P4) in order to estimate the relevant tensile forces and to make it possible a modification of the pre-loads of the hangers if they do not comply with design values.

Forces on deck-end ties are identified in P1, P2 and P3 as previously through the frequency identification while in P4 tests are not performed due to logistic problems. As for bearings, the relative displacements of the arch isolators are measured only during P4. The overall bridge dynamic behaviour is identified through AVTs performed during all the loading configurations of the static load test (P4) and finally, an impact load test is performed through a truck passing over a bump located on the arch span.

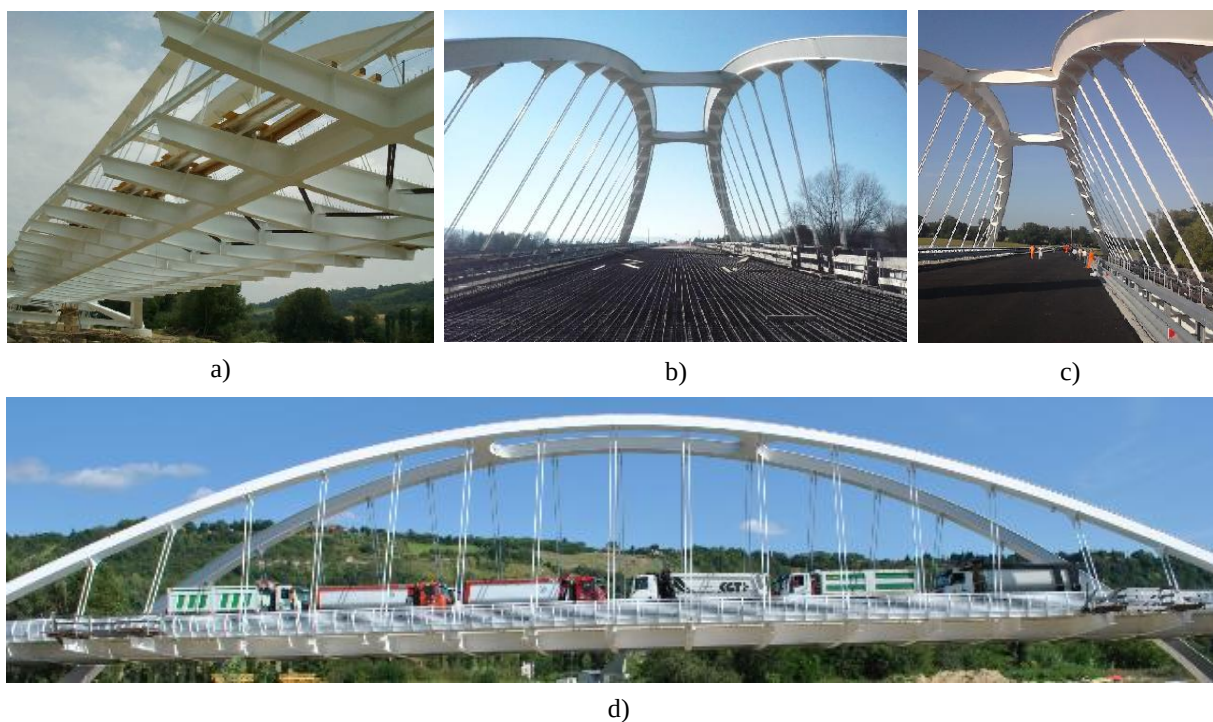


Figure 2. Construction phases: a) P1, b) P2, c) P3, d) P4.

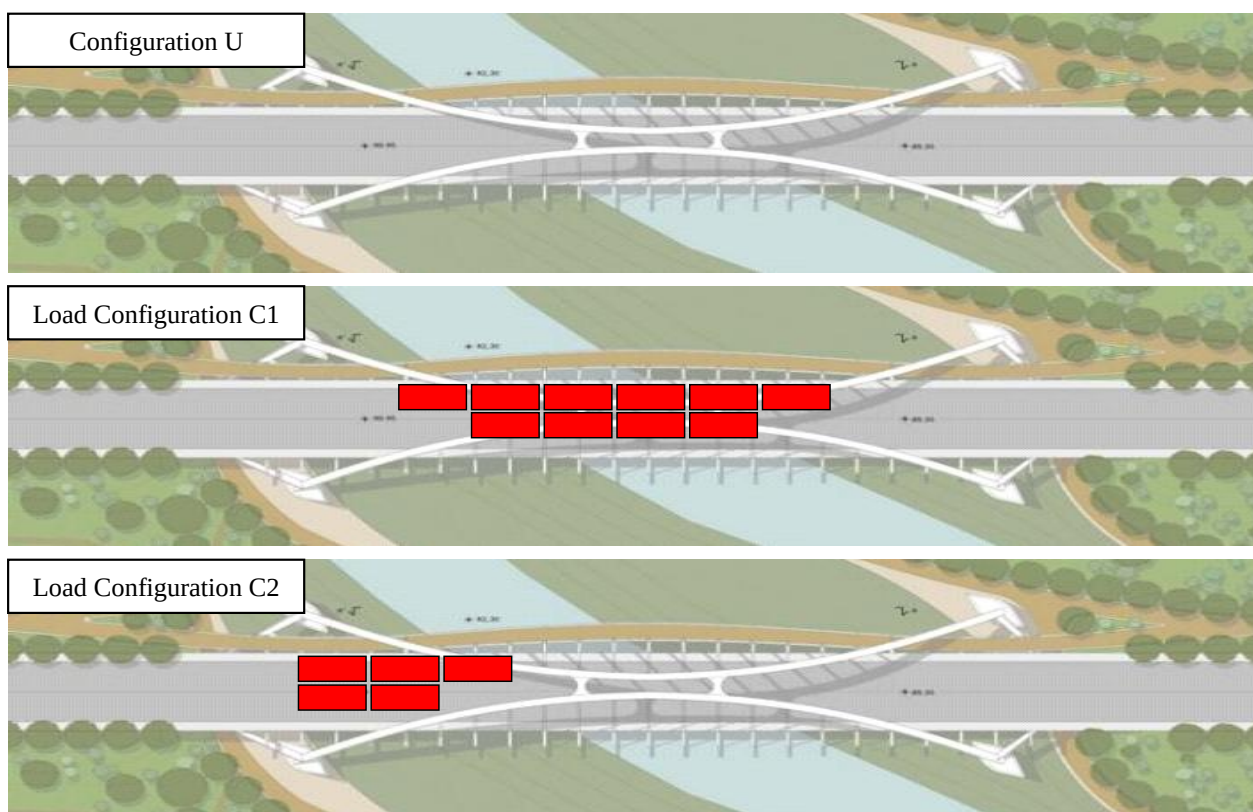


Figure 3. Load configurations during the static load test.

4. Tests on hangers and deck-end ties

The tensile forces acting on hangers and deck-end ties are estimated based on the results obtained from dynamic and static tests.

Dynamic tests consist in Impact Load Tests (ILTs) where the cable accelerations due to impulsive forces are measured; during tests natural frequencies are identified from which the relevant tensile forces are estimated starting from analytical expressions or relationships based on results of numerical models. This methodology is relatively simple, fast and rather cheap, but the tensile force estimation can be quite rough if the cable behaviour is far from the theoretical one (i.e. non-holonomic restraint conditions, non-linearities, lack of mechanical and geometrical homogeneity and uniformity, etc...). Impact tests on all the 60 hangers are performed in the construction phases P1, P2 and P3 while in P4 only a selected set of hangers are monitored due to the limited time available between the different loading configurations. In addition, being P4 composed of loading configurations U, C1 and C2, impact tests on the selected hangers are performed at different stages, according to the following sequence: *i*) at configuration U₀, *ii*) at configuration C1, *iii*) at the unloading from C1 (configuration U₁), *iv*) at configuration C2 and *v*) at the unloading from C2 (configuration U₂). Since ILTs only allow a rough tensile force estimation as a consequence of uncertainties due to the actual restraint conditions and mass distribution, in some hangers the latter are accompanied by strain gauge measurements that provide a direct information on the tensile forces acting on the hangers. Gauges are installed for the monitoring of forces on the hangers and deck-end ties on P4; given the complexity and time consuming for the sensor installation, strain gauges measurements are performed only on a reduced number of hangers and results are adopted to validate the values of tensile forces estimated with ILTs. Strain gauge measurements are performed during P4 following the same sequence adopted for impact tests.

4.1 Impact load tests

The measurement chain used to perform the ILTs is composed by a PCB impulse force test hammer model 086D20 with load cell and a rubber tip and by a piezoelectric accelerometer. The hammer, having mass of 1.1 kg, is characterised by a measurement range of ± 22240 N pk, sensitivity of 0.23 mV/N, and frequency range from 1 to 300 Hz. The accelerometer is characterized by a sensitivity of 98 mV/g, measurement range of 70 g pk and frequency range between 1 and 2000 Hz. The sensor and the hammer are connected by means of coaxial cables with BNC connectors to a 4-channels dynamic signal acquisition module NI-9234 with measurement range of 0.5 g pk, ADC resolution of 24 bits, signal ranges of ± 5 V and sample rate of 51.2 kS/s/ch mounted on a 1-slot USB chassis NI cDAQ 9171. To control the acquired signals during the tests and to store the data, a laptop

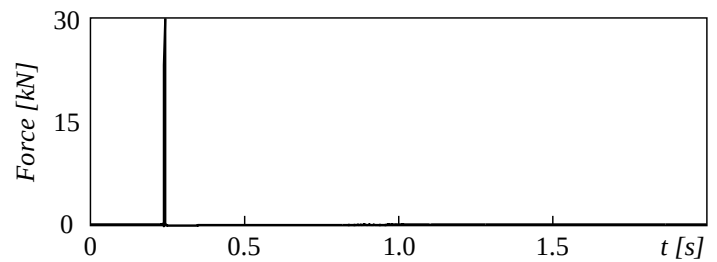
equipped with a dedicated software developed in Labview [33] environment is used.

The accelerometer is mounted on the hangers and the deck-end ties through a magnetic adapter at different positions during the same test in order to avoid the possibility that its position coincides with a node of one of the first vibration mode shapes (Fig. 4a). Taking into account the restraints conditions of the hangers and deck-end ties, the accelerometer is mounted to measure accelerations in the plane in which the steel member behaves as a pinned element. Three impulse forces are applied in the same plane for each test, by changing the hammer impact point in order to avoid exciting the hanger at mode shape nodes. Considering features of the used acquisition module (NI-9234), a sampling rate of 2048 Hz is used to achieve high resolution in time domain. An acquisition time duration of 10 s is considered to catch the entire duration of the element oscillation.

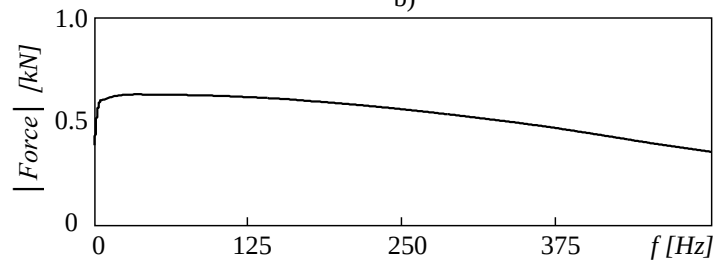
A typical time history of the impulse force recorded by the load cell of the hammer for a medium intensity hammer blow is reported in Fig. 4b, whereas the corresponding frequency spectrum is shown in Fig. 4c. The spectrum, characterized by a constant force level up to about 100 Hz that decreases to nearly zero at about 1000 Hz, allows the investigation of the dynamic behaviour of the steel element in a wide range of frequencies.



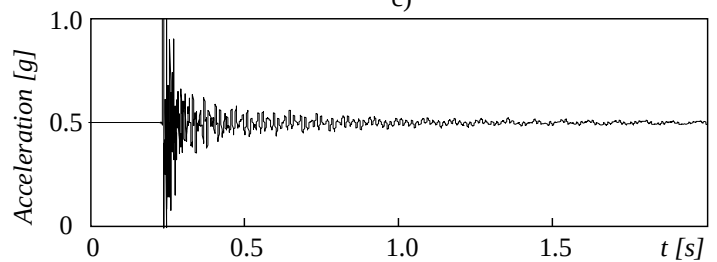
a)



b)



c)



d)

Figure 4. ILTs on hangers: a) execution, b) impact time history, c) impact frequency spectrum, d) acceleration time history.

The acceleration time history recorded by the sensor (Fig. 4d) shows a classical damped harmonic oscillation which tends to become null after a pair of seconds. The natural frequencies of the investigated elements (hangers and deck-end ties) are evaluated from the peaks of the Frequency Response Functions (FRFs) obtained from the ratio between the measured acceleration (output) and the impulse force (input). A typical FRF resulting from the tests shows several peaks which allow the identification of at least five natural frequencies. For the sake of brevity, only the results of three hangers (H1, H2 and H3 in Fig. 5) and two deck-end ties (T1 and T2 in Fig. 5) are reported (Fig. 5).

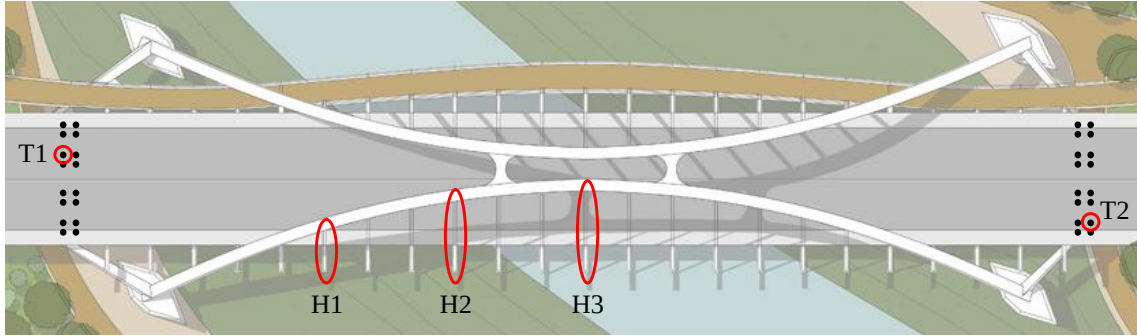


Figure 5. Hangers and deck-end ties considered as representative.

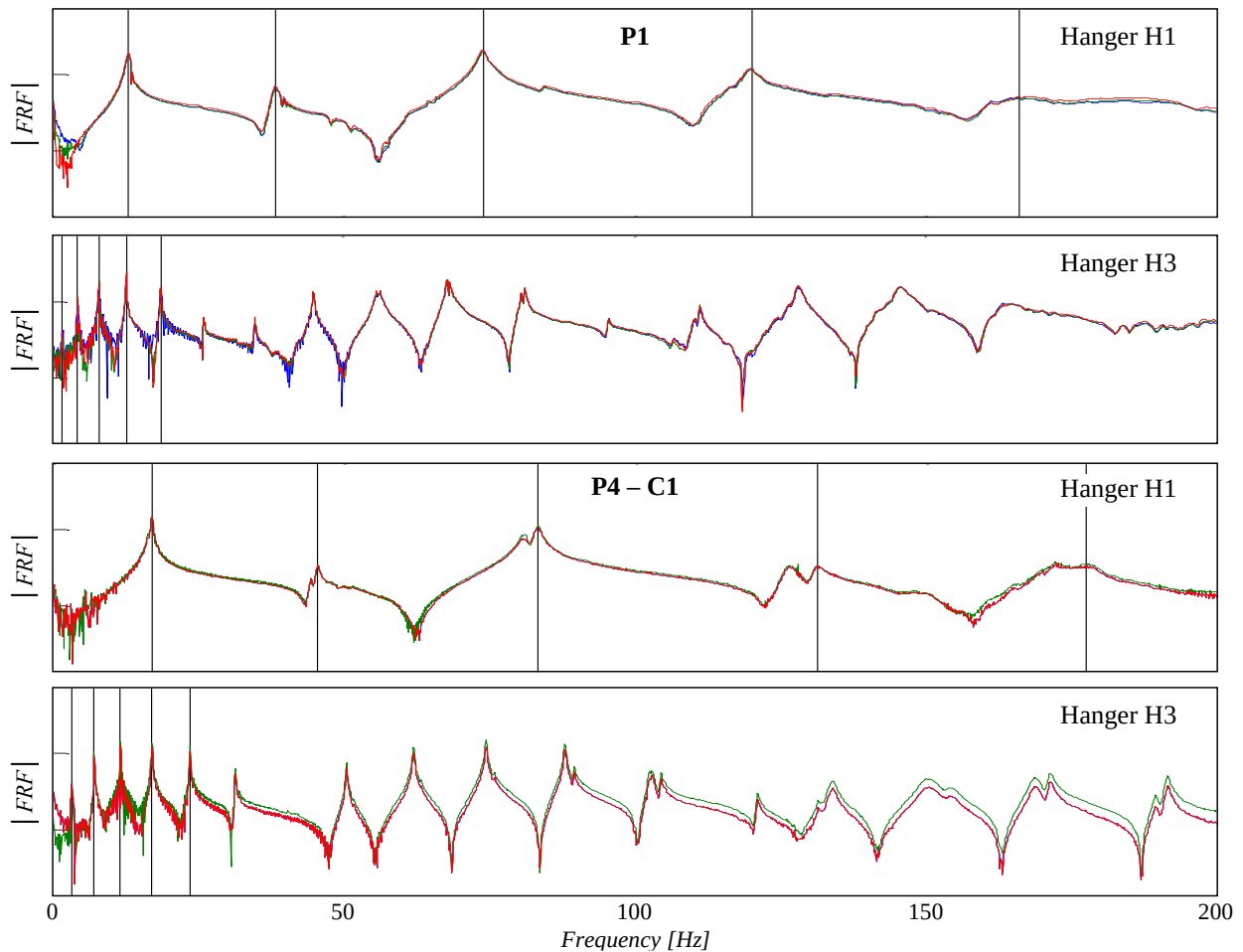


Figure 6. FRFs of H1 and H3 obtained for tests performed in P1 and in P4 with the maximum load on the deck (C1) (colours refer to different hammer blows).

The considered hangers are representative for short, medium and long cables, respectively, while a deck-end tie for each deck end is chosen as representative of the set. Strain gauges measurements will be performed on the same hangers; in this way a direct comparison of data from the two test typologies will be possible. For brevity, Fig. 6 shows the FRFs of H1 and H3 obtained for tests performed in P1 and in P4 (with the maximum load on the deck, C1). Clear peaks identifying the natural frequencies of the elements are always evident with the fundamental frequency of the short cable sensibly higher (about six times) than that of the long one. On the basis of test results during P1, pre-loads of some hangers were modified in order to adjust clearly anomalous situations and to obtain very similar frequencies for hangers belonging to the same couple. For the selected elements, frequency values identified from tests at different construction phases are all listed in Table 1. It is worth mentioning that results of tests may be slightly affected by the temperature, and particularly by a non-homogenous temperature of all the elements that, during tests, may be directly or not exposed to the sun. An overall increase of the frequency values is observed, as expected, by increasing the loads on the deck (i.e. by increasing the axial force in the element). As for P4, an increase of about 7, 12 and 15% is observed for frequency values of the three hangers, relevant to configuration C1 with respect to configuration U; at the deck unloading, frequencies almost come back to the original values. On the other hand, frequencies are only slightly affected by loads of configuration C2 (variation are of the order of 3÷4%), which are far away from the investigated elements.

The cable tensile forces are estimated with an identification process based on difference minimization between numerical and experimental natural frequencies of the hangers. Numerical models are developed in standard software for structural analysis taking into account the actual geometry and mass distribution of the elements. As an example, in Table 2 the estimated tensile forces for hangers H1, H2 and H3 are shown for P3 (i.e. at the end of the bridge construction) and for P4 and configuration C1. Forces estimated from dynamic tests are compared with the design values (bridge subjected to gravity loads – P3) and with the numerical prediction of tensile forces obtained from the design model for load configuration C1. It is worth mentioning that the estimated values for phase P3, required by designers to check the tensile forces in the hangers, have been accepted as reasonable for the unloaded condition although not perfectly compliant with design values. In this sense, a comparison of the design and estimated forces is of minor relevance while increments due to added loads, resulting from both experimental and numerical methodologies, may be important for testing the model reliability. During the static load tests an increase of 40, 80 and 60 kN are observed experimentally with respect to the unloaded condition; the latter agree quite well with the numerical predictions.

Table 1. Hanger and deck-end tie natural frequency values.

Phase	Element	Frequency [Hz]				
		$f1$	$f2$	$f3$	$f4$	$f5$
P1	H1	13.2	38.4	74.0	120.1	165.9
	H2	3.1	7.3	13.4	21.6	32.7
	H3	2.0	4.7	8.3	13.0	19.0
	T1	12.1	30.0	60.1	95.2	137.1
	T2	26.3	81.5	144.3	233.0	293.1
P2	H1	13.6	39.2	75.3	120.8	188.9
	H2	3.6	8.3	14.7	23.0	34.1
	H3	2.4	5.4	9.3	14.1	20.2
	T1	9.0	25.8	54.9	90.8	133.9
	T2	18.2	71.5	124.4	199.8	292.2
P3	H1	17.0	45.2	82.7	130.0	174.9
	H2	5.0	10.8	17.9	27.0	38.5
	H3	3.5	7.3	11.8	17.2	23.7
	T1	15.0	35.1	66.6	103.7	144.5
	T2	31.1	91.3	158.3	250.3	374.8
P4 – U₀	H1	16.9	45.3	82.9	131.1	174.6
	H2	5.0	10.8	17.9	27.1	38.6
	H3	3.4	7.3	11.8	17.3	23.8
P4 – C1	H1	18.0	47.1	85.0	132.7	179.8
	H2	5.6	11.8	19.3	28.6	40.4
	H3	3.9	8.2	13.0	18.6	25.4
P4 – U₁	H1	17.1	45.6	83.3	131.2	177.4
	H2	5.0	10.7	17.9	26.9	38.4
	H3	3.5	7.3	11.8	17.2	23.8
P4 – C2	H1	17.7	46.6	84.5	132.1	178.7
	H2	5.0	10.8	18.0	27.1	38.6
	H3	3.5	7.4	11.9	17.4	23.9
P4 – U₂	H1	17.2	45.6	83.5	131.5	177.2
	H2	5.0	10.7	17.9	27.1	38.5
	H3	3.4	7.3	11.8	17.2	23.8

Table 2. Estimated and design hanger tensile forces for P3 and configuration C1 in P4.

Phase	Element	Tensile force [kN]		Estimated ΔN [kN]	Design ΔN [kN]
		Estimated	Design		
P3	H1	160	116.5	/	/
	H2	220	139.3	/	/
	H3	200	131.5	/	/
P4 – C1	H1	200	154.2	40	37.7
	H2	300	191.7	80	52.4
	H3	260	185.5	60	54.0

4.2 Strain gauge measurements

The instrumentation adopted for the strain measurements of hangers is composed by rectangular rosette strain gauges with pre-attached ready-to-use cables model Vishay C2A-06-125LR-350 and tee rosette strain gauges with pre-attached ready-to-use cables model Vishay C2A-06-125LT-350, both characterized by 350 ohms ($\pm 0.6\%$) and gage factor of 2.09 ($\pm 0.5\%$). Moreover, a signal conditioner and converter model HBM Spider 8 and a laptop, are adopted to acquire signals of strain gauges. The sensors installation (Fig 7a) over the hangers is performed according to the following steps: *i*) gluing surface preparation by exterior paint removal and accurate cleaning with solvent, *ii*) strain gauge gluing and final cleaning with alcohol pads, *iii*) measuring system protection (strain gauge and cable connections) with two polyurethane lacquer layers and through the protective ABM75, composed by mastic and aluminium layers. Strain gauges are installed in H1, H2 and H3 (Fig. 4) adopting a full-bridge configuration (Fig 7a) using two rectangular rosette strain gauges positioned exactly opposite on the same point of the cable. The full-bridge configuration allows the exclusion of temperature and self-weight contributions to the measure of strains. It worth mentioning that sensor application is based on the assumption that strains are constant over the cross-section, neglecting accidental bending moments.

The instrumentation adopted for the strain measurements of the deck-end ties is composed by vibrating wire strain gauges model GST-BVC with rope length of 153 mm, measurement range of 3000 μs , sensitivity of 1 μs and resonant frequency of 800 Hz. A portable readout unit is adopted to acquire the signals. Two sensors are installed in the middle of both tie T1 and T2 (Fig. 7b), also monitored through dynamic tests, in order to get a mean more reliable value of the measured strains.

Strain gauge measurements are performed during P4 in both the loaded and unloaded configurations, following the sequence already presented for impact tests. Table 3 shows the variations of the strains obtained at different stages of P4 with respect to the unloaded condition as well as the relevant estimated variation of the tensile force evaluated assuming the Young's modulus of the material equal to $E_s = 210000 \text{ MPa}$. Positive strain values refer to the gauge elongation. As for hangers, for load configuration C1, the tensile force variations follow the expected trend, with the hangers located close to the mid-span presenting a higher strain variation with respect to edge ones. Similar considerations hold in the case of load configuration C2 for which hangers located near the loaded support show a greater strain variation with respect to the others. After the load removal (configuration U after C1 and after C2), strains return almost to zero. It is worth observing that the tensile force increment obtained from static test (P4, load condition C1) is comparable with that resulting from impact test, reposted in Table 2.

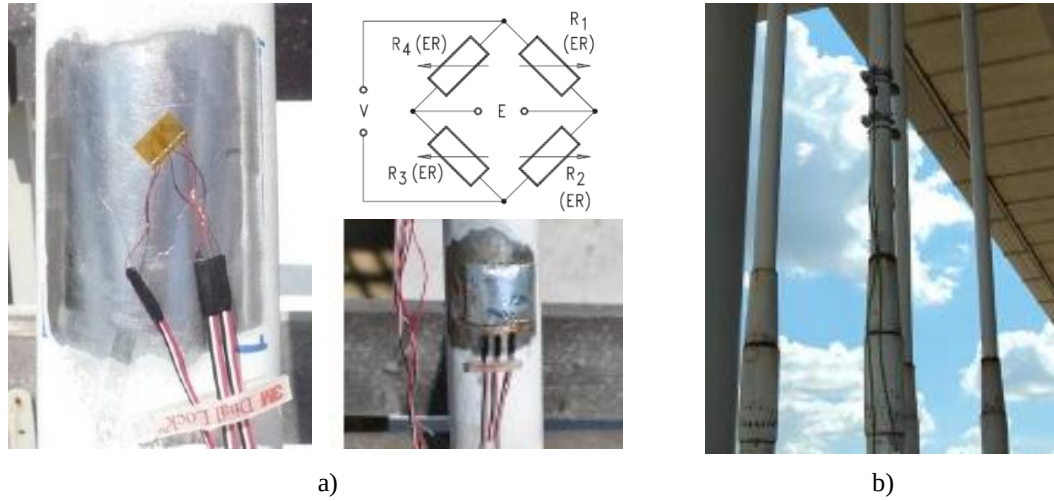


Figure 7. Strain gauge measurements: a) rosette strain gauges, b) vibrating wire strain gauges.

Table 3. Strain and tensile force variations on hangers and deck-end ties at different stages of P4.

Element	U ₀		C1		U ₁		C2		U ₂	
	ϵ [μ s]	ΔN [kN]	ϵ [μ s]	ΔN [kN]	ϵ [μ s]	ΔN [kN]	ϵ [μ s]	ΔN [kN]	ϵ [μ s]	ΔN [kN]
H1	0.00	0.00	63.89	43.16	1.16	0.79	34.52	23.32	-9.61	-6.49
H2	0.00	0.00	82.51	55.74	-0.93	-0.63	2.91	1.97	-4.67	-3.16
H3	0.00	0.00	82.33	55.62	-0.3	-0.20	5.37	3.63	-0.56	-0.38
T1	0.00	0.00	63.07	42.61	48.84	33.00	48.13	32.52	41.03	27.72
T2	0.00	0.00	4.88	3.30	-8.53	-5.76	16.50	11.14	29.98	20.25

5. Tests on seismic isolators

During P4 relative displacements of the HDRBs are measured to get the shear deformation of the device during the loading and unloading stages. In detail, relative displacements of the four Seismic Isolators (SI1, SI2, SI3 and SI4) (Fig. 8) positioned at the arch bases are monitored using linear potentiometer displacement transducers (model GST-TP) characterized by measurement range of 50 mm, diameter of 16 mm and repeatability of 0.01 mm. Signals are acquired with a portable readout unit model GST-CL with 14-bit A/D converter. For each SI, three transducers (H1, H2 and H3) are placed according to the layout of Fig. 8 to measure horizontal displacements in two principal directions and to capture possible rotations about the vertical axis. In addition, four transducers (V1, V2, V3 and V4) are installed for each SI to measure the relative vertical displacements and rotations between the upper and lower plates at each corner.

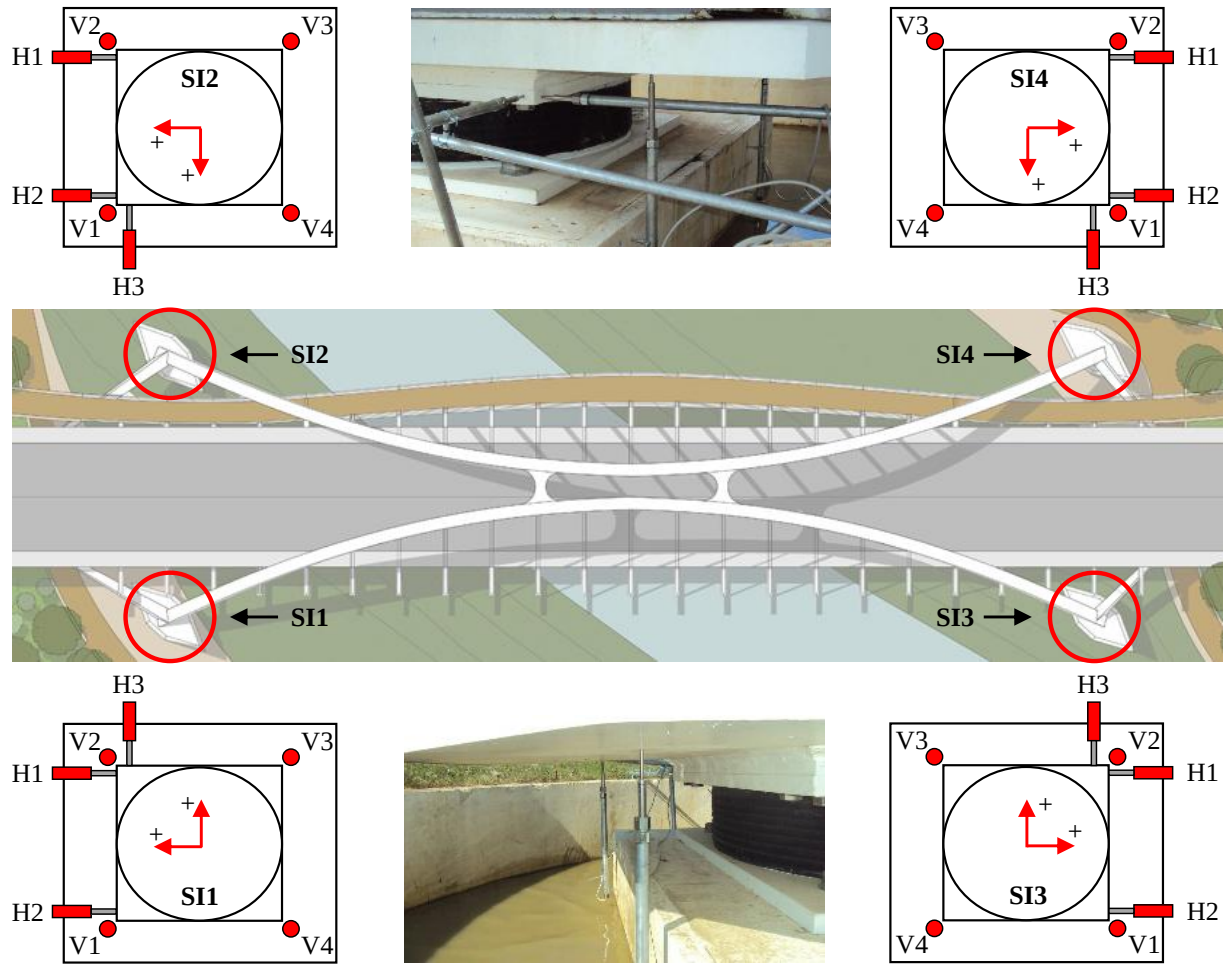


Figure 8. Tested SI and tests layout.

Table 4 shows the SI relative displacements $H\#$ and $V\#$ measured during the stages in P4; positive displacement values refer to the transducer shortening. Taking into account the sensors layout for the different SI, it can be deduced that loads produce the arch deflection with isolators deforming in opposite directions in the arch planes and producing an enlargement of the arch base of about 4 mm for configuration C1 and 1.5 mm for configuration C2. Torsional movements of isolators are almost null while rotations, deduced by vertical transducers are in line with the arch deflection.

6. Dynamic identification of the overall structure

During P4, AVTs are repeated 3 times in order to identify the modal parameters of the arch bridge at different stages: the first test is performed immediately before the first loading phases (i.e. at the unloaded configuration U_0) while the second and third tests are executed during the loading phases of P4 (load configuration C1 and C2). In addition, an impact test is performed on the unloaded bridge at the end of P4 (U_2) with a truck passing over a bump placed over the bridge deck in proximity of the mid-span.

Table 4. Vertical and horizontal SI relative displacements measured in P4.

Seismic isolator	Transducer	Displacements [mm]				
		U_0	C1	U_1	C2	U_2
SI1	H1	0.00	2.32	0.16	1.38	0.18
	H2	0.00	2.32	0.23	1.20	0.23
	H3	0.00	-0.43	0.14	0.26	0.13
	V1	0.00	0.21	0.05	0.50	0.10
	V2	0.00	-0.11	-0.16	-0.03	-0.28
	V3	0.00	0.00	-0.27	-0.53	-0.39
	V4	0.00	0.32	-0.01	0.03	-0.01
SI2	H1	0.00	3.34	-1.02	1.65	-1.07
	H1	0.00	1.97	-2.29	0.16	-2.33
	H1	0.00	-1.28	0.14	-1.02	0.28
	V1	0.00	-8.54	-0.26	0.63	-0.15
	V2	0.00	0.16	0.01	0.09	-0.08
	V3	0.00	-0.14	0.02	-0.53	-0.15
	V4	0.00	0.25	-0.21	0.14	-0.16
SI3	H1	0.00	3.59	1.22	1.64	0.93
	H1	0.00	3.61	1.29	1.76	1.04
	H1	0.00	-0.37	0.25	0.13	0.18
	V1	0.00	0.23	0.11	-0.03	0.11
	V2	0.00	-0.12	-0.11	-0.22	-0.25
	V3	0.00	0.15	-0.18	-0.06	-0.31
	V4	0.00	0.54	0.08	0.16	0.19
SI4	H1	0.00	5.83	1.51	1.89	0.89
	H1	0.00	5.90	1.62	2.05	1.02
	H1	0.00	-1.95	-0.60	-0.51	-0.49
	V1	0.00	0.79	0.07	-0.01	0.06
	V2	0.00	-0.04	0.04	-0.10	-0.21
	V3	0.00	-0.32	-0.07	-0.06	-0.26
	V4	0.00	0.48	-0.02	0.03	-0.02

Accelerations during tests are registered with eleven low noise uniaxial piezoelectric PCB 393B31 accelerometers with ceramic flexural ICP, sensitivity of 10 V/g, broadband resolution of 1 μ g rms, measurement range of 0.5 g pk and frequency range between 0.1 and 200 Hz. Sensors are connected by means of coaxial cables with BNC connectors to three 4-channels dynamic signal acquisition module NI-9234 characterized by measurement range of 0.5 g pk, ADC resolution of 24 bits, signal ranges of ± 5 V and sample rate of 51.2 kS/s/ch, mounted on 8-slot USB chassis NI cDAQ 9178. A laptop equipped with a dedicated software developed in Labview environment [23] is adopted to acquire signals and to store the data. Accelerometers are located on the bridge deck according to the layout of Fig. 9 in order to capture horizontal and vertical displacements of a significant number of points to identify the flexural and torsional behaviour of the deck. For each test,

2000 s long records with sampling rate of 2048 Hz are acquired; this time length provides enough data in order to obtain modal parameters with a good accuracy, as reported in [34] where a length of the time window acquired of about 1000-2000 times the period of the fundamental mode is recommended.

Operational Modal Analyses (OMA) are performed on the basis of the acceleration measurements during the AVTs. The ambient excitation (wind, microtremors, etc...) is not controlled and is assumed to have a flat spectrum such as a white noise; therefore, modal parameters of the bridge are identified through the Covariance-driven Stochastic Subspace Identification (SSI-COV) output-only technique [35-36], which works in time domain. According to the SSI-COV technique, a mode is considered to be consistent with respect to frequency, damping ratio and mode shape when, increasing the model order one by one, it shows a natural frequency variation lower than 1%, a damping ratio variation lower than 2%, and a Modal Assurance Criterion (MAC) between the modes greater than 0.98, respectively. MAC is an index [37] that expresses with a numerical value ranging between 0 and 1 the matching between two mode shapes (0, no matching, 1 perfect match).

Natural frequencies obtained from the OMA during the different load configurations of P4 are listed in Table 5. In detail, 10 vibration modes are always identified, corresponding to 4 bending, 4 torsional and 2 transverse vibration modes of the deck. Frequencies obtained from tests during load configuration C1 are the lower ones, due to the presence of the maximum mass on the deck (truck masses) positioned close to the mid-span of the bridge; frequencies obtained for load configuration C2 are higher but closer to those associated to the bridge unloaded condition (U_0) because the added mass due to trucks is less important with respect to that of C1 and located near a bridge support. Finally, it can be observed that frequencies obtained for U_2 are close to those relevant to U_0 , confirming the linear elastic behaviour of the bridge subjected to the proof loads.

Table 5. Experimental natural frequencies of the bridge in U_0 .

Mode n.	Mode name	Frequency [Hz]				
		U_0	C1	C2	U_2	Impact
1	1 st bending	1.18	1.05	1.13	1.17	1.16
2	1 st transverse	1.45	1.75	1.45	1.45	1.40
3	2 nd bending	1.88	1.42	1.90	1.86	/
4	3 rd bending	2.22	2.09	2.07	2.19	2.17
5	1 st torsion	2.43	2.37	2.41	2.43	/
6	2 nd transverse	2.81	2.69	2.74	2.75	2.79
7	4 th bending	3.27	2.72	2.94	3.23	/
8	2 nd torsion	3.33	3.01	3.35	3.30	3.27
9	3 rd torsion	4.07	4.20	4.08	4.01	/
10	4 th torsion	5.16	5.21	5.18	5.13	5.06

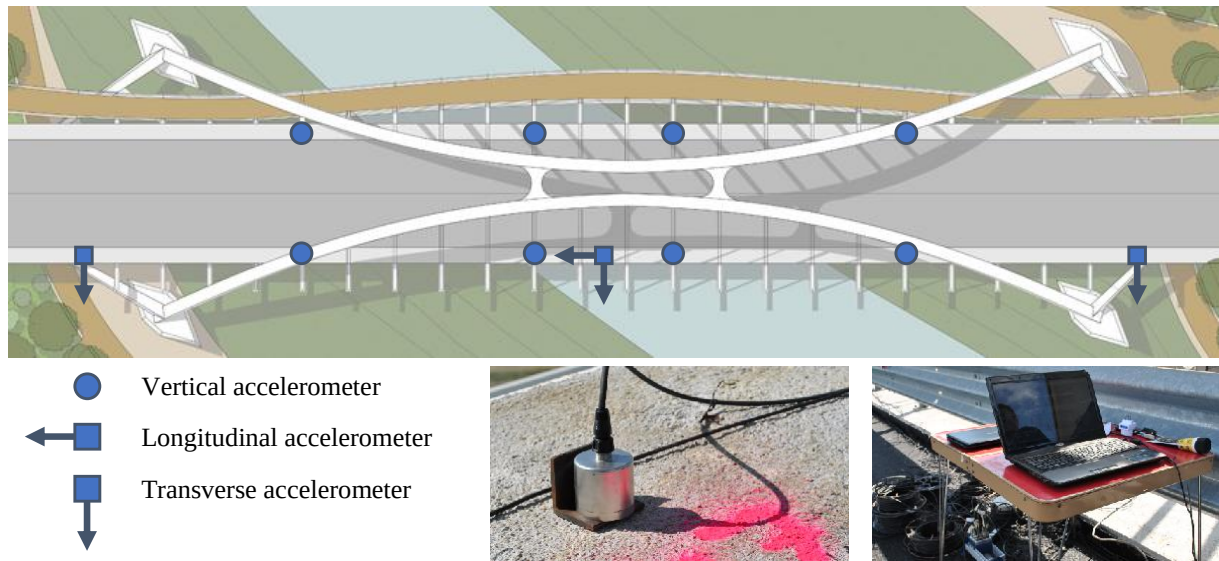


Figure 9. Accelerometer array on the bridge deck.

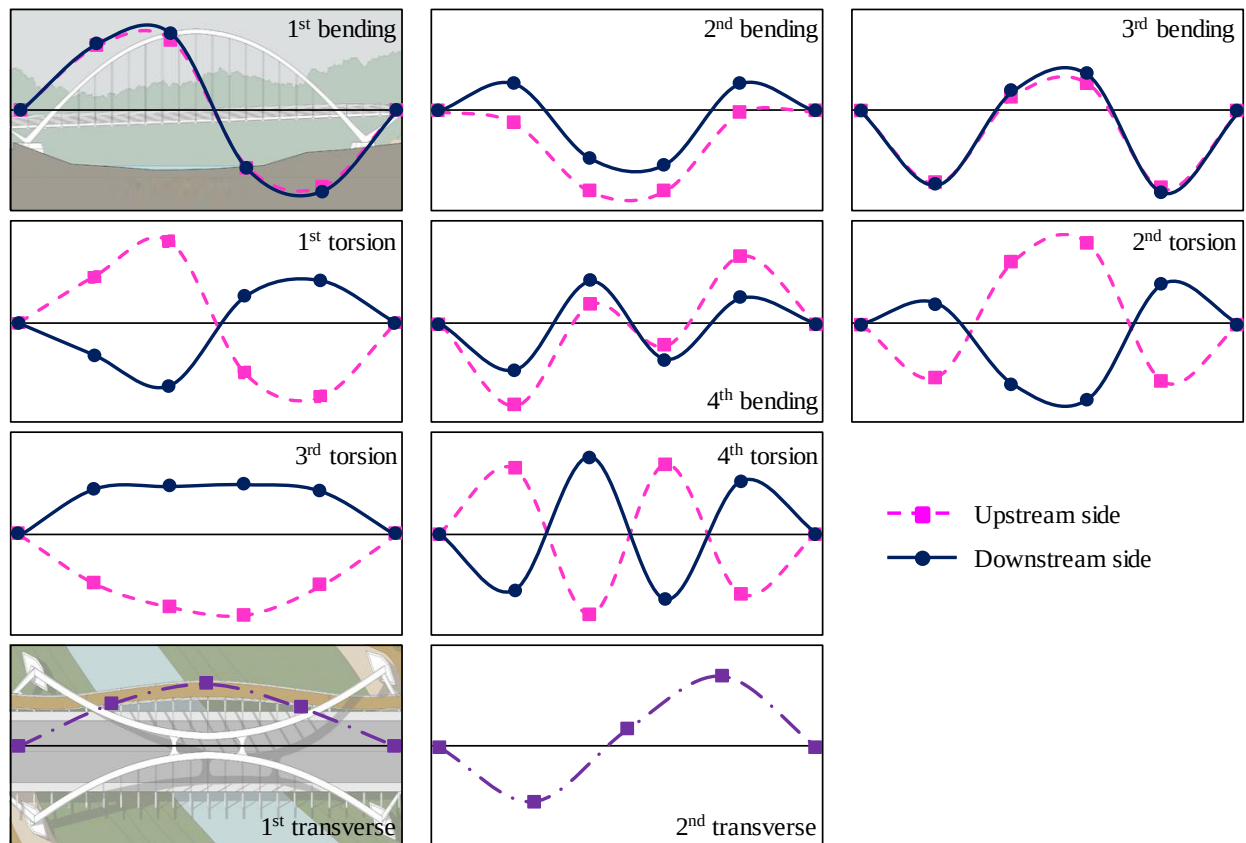


Figure 10. Identified mode shapes for the configuration U_0 in P4.

As for the impact test, performed with a truck passing over a bump located on the deck, only 6 vibration modes are identified using the SSI-COV methodology (an output-only technique is also used in this case because the input force is unknown) characterised by frequency values similar to those achieved in unloaded configuration U_0 .

Fig. 10 shows the identified mode shapes relevant to the unloaded configuration (U_0); similar

mode shapes are obtained for tests performed during load configuration C1 and C2 as well as for the impact test.

7 Tests interpretation through numerical modelling

A Finite Element (FE) model of the whole bridge is developed in SAP2000® [38] in order to provide interpretation of main results of the experimental tests carried out on the bridge at his final state. Steel members of arches and the transverse beams are modelled with straight elastic frame elements by adopting a suitable mesh for curved members and variable cross sections. Hangers are modelled with frames with uniform cross-section taking into account the actual restraint conditions at the ends. Braces placed between the main longitudinal girders due to stability issues during construction, are also modelled to automatically include the relevant mass through the software potentials. All beam elements are positioned in their relevant centroids using rigid links at the end nodes when misalignments among elements are deemed important. The R.C. deck slabs of the bridge, of the approaching viaducts and of the pedestrian walkway are modelled by means of shell elements having different thickness and located in the midplane of the slabs; the vertical eccentricity between the steel girders and the shell planes is taken into account through the use of rigid links.

Piers of approaching viaducts are not modelled since they are squat and therefore their contribution on the bridge overall deformability is assumed to be negligible considering that the structure is seismically isolated. HDRBs are modelled with elastic links located at the arch bases and at the deck intermediate supports. Masses of structural elements are automatically included in the model while masses of non-structural elements are suitably added considering their actual distribution. Fig. 11 shows some pictorial views of the developed FE model.

The base model above described, which is assumed to be representative of the structure at the end of construction, has been slightly modified depending on the test results to be interpreted: for addressing results of AVTs performed in P4 at the unloaded condition U_0 , the concrete stiffness of the slabs is calculated considering the uncracked condition and increasing the mean value of the elastic modulus (of about 10%) to take into account the dynamic properties of the material (subjected to very low amplitude vibrations). In addition, the stiffness contribution of the road pavement is considered in a simplified way by further increasing the elastic modulus of the concrete slabs. To investigate the loaded bridge response during P4 in configuration C1 and C2, the FE model is modified to include the truck loads (and the relevant masses). In this section the truck is modelled as a lumped mass rigidly connected to the deck whereas the dynamic truck-bridge interaction will be studied in the following section.

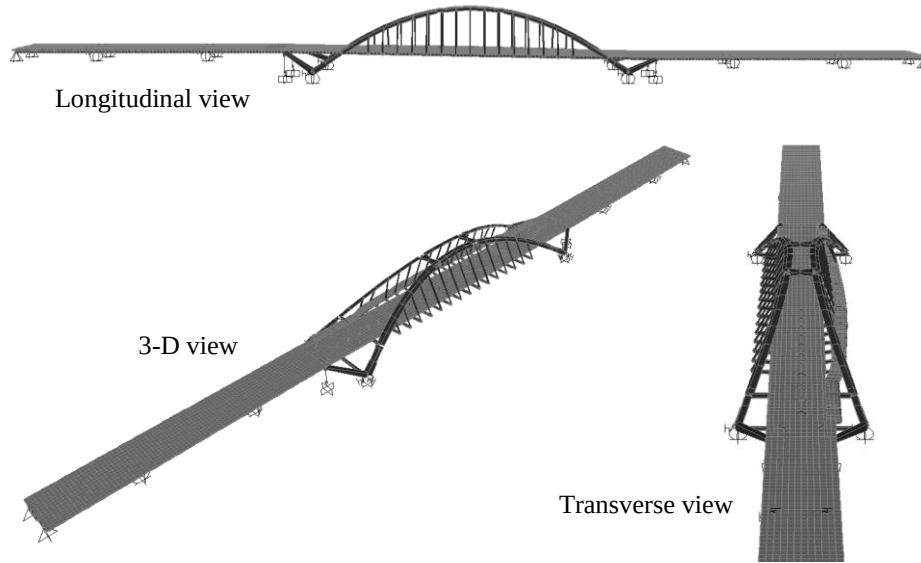


Figure 11. Pictorial views of the bridge FE model.

In detail, each truck is modelled as a lumped mass of approximately 36 t rigidly connected to the bridge deck in the position of the truck centroid for both configuration C1 and C2. In the sequel, it will be shown that the truck-bridge interaction may be important in the interpretation of dynamic AVTs; indeed, truck are dynamic systems that can be modelled as masses supported on deformable elements whose stiffness represents the contribution of the truck shock absorbers and tires. This problem will be analytically addressed for a simple case in the sequel, providing some useful considerations for the interpretation of test results through a more rigorous FE model considering the truck-bridge interaction.

7.1 Tensile forces in hangers

Increment of tensile forces in hangers due to truck loads of configuration C1 and C2 are computed from the FE model and compared with the experimental ones obtained from strain gauges measurements. Comparisons are reported in Table 6 for the previous selected hangers (H1, H2 and H3); the numerical model is capable to capture the force increments with sufficient accuracy (the percentage errors are 13, 6 and 3% for hangers H1, H2 and H3, respectively) while for configuration C2 variations between numerical predictions and experimental results are greater in percentage (even if it should be remarked that absolute values are low, especially for H2 and H3).

Table 6. Comparison between experimental (strain gauges) and numerical (FE model) tensile force increments for configurations C1 and C2 in P4.

Hanger	Tensile force increment ΔN [kN]			
	C1		C2	
	Experimental	Numerical	Experimental	Numerical
H1	43.16	37.71	23.27	33.21
H2	55.74	52.38	1.97	6.69
H3	55.62	54.03	3.63	4.24

7.1 Ambient vibration tests

The modal parameters of the whole bridge, obtained from a conventional eigen value analysis, are compared with results of OMA. Table 7 compares the numerical frequency values with the experimental ones; for tests performed during configuration C1 and C2, the truck-bridge interaction is herein neglected. For load configuration U_0 the numerical frequencies are in good agreement with the experimental ones, particularly for bending modes for which the percentage mean error of the numerical frequency is about 1%. For configurations C1 and C2, experimental and numerical bending mode frequencies highlight higher differences than those relevant to the unloaded configuration U_0 . In detail, for C1, the mean percentage error of the numerical frequency of bending modes increases to about 8%, with a maximum error of about 17% for the 2nd bending mode. As for transverse natural frequencies, which overall present higher differences between numerical and experimental values, the percentage mean error of the first two numerical frequencies is about 4% for configuration U_0 and about 10% for configuration C1. Furthermore, it is worth noting that the experimental first transverse frequency relevant to C1 increases with respect to that of U_0 , contrarily to what expected in view of the increased mass: this can be explained considering the truck-bridge interaction phenomena, as discussed in the next section. For C2, frequencies much similar to that obtained for U_0 are obtained, consistently with the added mass position. Finally, independently on the loading condition, the model is not able to capture torsional modes very well, probably due to warping torsional effects that are not included in the model.

Numerical and experimental mode shapes are compared by means of the MAC. Fig. 12 portrays the MAC values between the experimental and numerical mode shapes for the unloaded configuration U_0 and for both the loaded configurations C1 and C2. The MAC is calculated considering transverse and vertical displacements of monitored points to compute the index relevant to transverse and bending modes (including torsional), respectively. The corresponding modes (diagonal entries) have almost always high MAC values, indicating a very good agreement between experimental and numerical mode shapes.

Table 7. Bridge experimental and numerical frequencies for configurations U₀, C1 and C2 in P4.

Mode n.	Mode name	Frequency [Hz]					
		U ₀		C1		C2	
		Exp.	Num.	Exp.	Num.	Exp.	Num.
1	1 st bending	1.18	1.18	1.05	1.03	1.13	1.15
2	1 st transverse	1.45	1.57	1.75	1.45	1.45	1.54
3	2 nd bending	1.88	1.94	1.42	1.66	1.90	1.93
4	3 rd bending	2.22	2.20	2.09	2.10	2.07	2.10
5	1 st torsion	2.43	3.07	2.37	2.98	2.41	3.06
6	2 nd transverse	2.81	2.81	2.69	2.76	2.74	2.78
7	4 th bending	3.27	3.21	2.72	3.05	2.94	3.00
8	2 nd torsion	3.33	3.66	3.01	3.46	3.35	3.63
9	3 rd torsion	4.07	4.25	4.20	4.16	4.08	4.22
10	4 th torsion	5.16	5.49	5.21	5.32	5.18	5.45

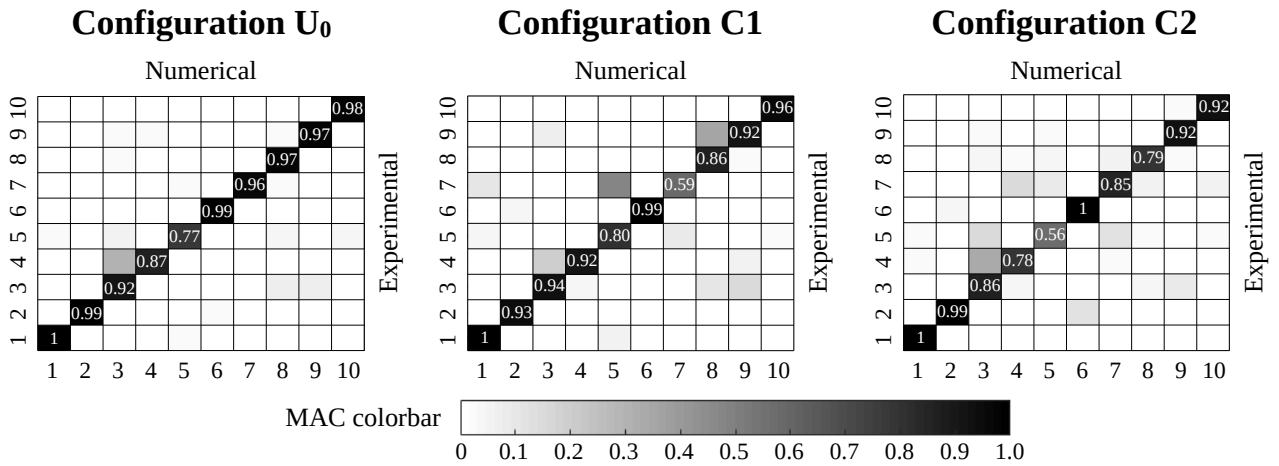


Figure 12. MAC values between experimental and numerical mode shapes for configuration U₀, C1 and C2.

8 Truck-Bridge dynamic interaction

In order to understand trends observed in the dynamic response of the bridge loaded by trucks, a deeper insight of the problem is carried out by considering a simple analytical model addressing the truck-bridge interaction problem. In detail, the bridge and the truck are reduced to the 2 Degree Of Freedom system (2DOF) depicted in Fig. 13, where k_2 and m_2 are the stiffness and mass associated to the i -th vibration mode considered for the bridge, and k_1 and m_1 are the stiffness and mass of the trucks; as already stated the truck stiffness is due to the combined contribution of shock absorbers and tires. By defining with

$$\omega_1 = \sqrt{\frac{k_1}{m_1}} \quad (1a)$$

$$\omega_2 = \sqrt{\frac{k_2}{m_2}} \quad (1b)$$

the fundamental circular frequency of the trucks and the bridge independently considered, the eigenvalue problem of the 2DOF system can be expressed in the following non-dimensional form [39]

$$\left(\begin{bmatrix} 1 & -1 \\ -1 & 1 + \frac{\omega_2^2 m_2}{\omega_1^2 m_1} \end{bmatrix} - \frac{\omega^2}{\omega_1^2} \begin{bmatrix} 1 & 0 \\ 0 & \frac{m_2}{m_1} \end{bmatrix} \right) \begin{bmatrix} \Phi_2 \\ \Phi_1 \end{bmatrix} = \mathbf{0} \quad (2)$$

where ω is the circular frequency of the 2DOF system and Φ_1 and Φ_2 are components of the eigenvector associated to the non-dimensional eigenvalue

$$\lambda = \frac{\omega^2}{\omega_1^2} \quad (2)$$

Furthermore, by defining

$$\bar{\omega} = \frac{\omega_2}{\omega_1} \quad (3a)$$

$$\bar{m} = \frac{m_2}{m_1} \quad (3b)$$

the following condition on the non-dimensional eigenvalue provides, as well known, a non-zero solution of the problem (2)

$$\det \left(\begin{bmatrix} 1 & -1 \\ -1 & 1 + \bar{\omega}^2 \bar{m} \end{bmatrix} - \lambda \begin{bmatrix} 1 & 0 \\ 0 & \bar{m} \end{bmatrix} \right) = 0 \quad (4)$$

which allows expressing analytically the non-negative non-dimensional circular frequencies of the 2DOF system as follows

$$\frac{\omega}{\omega_1} = \sqrt{\frac{1 + \bar{m}(1 + \bar{\omega}^2) \pm [(1 + \bar{m}(1 + \bar{\omega}^2))^2 + 4\bar{m}^2 \bar{\omega}^2]^{0.5}}{2\bar{m}}} \quad (5)$$

Fig. 13 plots Equation (5) for a fixed value of $\bar{m} = 0.16$; for $\bar{\omega}$ approaching 0 (i.e. the stiffness of trucks is much higher than the effective stiffness of the bridge relevant to the i -th vibration mode) one of the circular frequency approaches the null value, while the other tends to that corresponding to a Single Degree Of Freedom System (SDOF) having stiffness k_2 and mass $m_1 + m_2$. This hypothesis is the same adopted in modelling masses due to trucks on the previous described numerical model. For $\bar{\omega} = 1$ the classical response of a tuned system [40] is obtained while for $\bar{\omega}$ approaching infinite (i.e. the stiffness of trucks is much lower than the effective stiffness of the bridge relevant to the i -th vibration mode), one circular frequency tends to infinite while the other tends to ω_2 . This simple system can be used to interpret experimental tests performed at loading configurations C1 and C2 considering that measures are generally performed at the deck level so that fundamental frequencies

can be experimentally captured only if accelerations (and displacements) due to the modal vibration are sufficiently higher than the sensor noise. To better address this issue, Fig. 13 also shows the amplitude and phase of qualitative Frequency Response Functions (FRFs) of displacements of the two masses obtained by applying a steady-state unit displacement at the base of the system, and considering three different conditions (i.e. $\bar{\omega} < 1$; $\bar{\omega} = 1$; $\bar{\omega} > 1$). In addition, the expected FRFs resulting by only considering the added mass of the truck is reported with a grey dashed line for the same situations. Taking into account that frequencies are estimated from the deck response (green lines of Fig. 13), for $\bar{\omega} < 1$, the circular frequency estimated by neglecting the truck-bridge interaction (grey line) is higher than the actual one (that can be estimated experimentally) while for value $\bar{\omega} > 1$ the circular frequency estimated neglecting the truck-bridge interaction is lower.

For $\bar{\omega} = 1$ the experimental response shows two peaks of similar amplitude characterised by the same mode shape of the bridge, which may generate misunderstandings in the identification of the first frequency. It is worth mentioning that the frequency estimated by neglecting the truck-bridge interaction falls between the two previous mentioned frequencies; thus, the interpretation of the test may lead to conclude that the bridge stiffness is higher or lower than the expected one. Above aspects are important, also for condition $\bar{\omega} \geq 1$ because they may lead to conclude during final tests on bridges that the real structure is more deformable than the numerical model developed for the prediction and judgment of the proof test results.

Taking into account the problem described above, the truck-bridge dynamic interaction for the investigated bridge is investigated numerically by including trucks in the structural FE model not only in terms of masses, but also considering their stiffness.

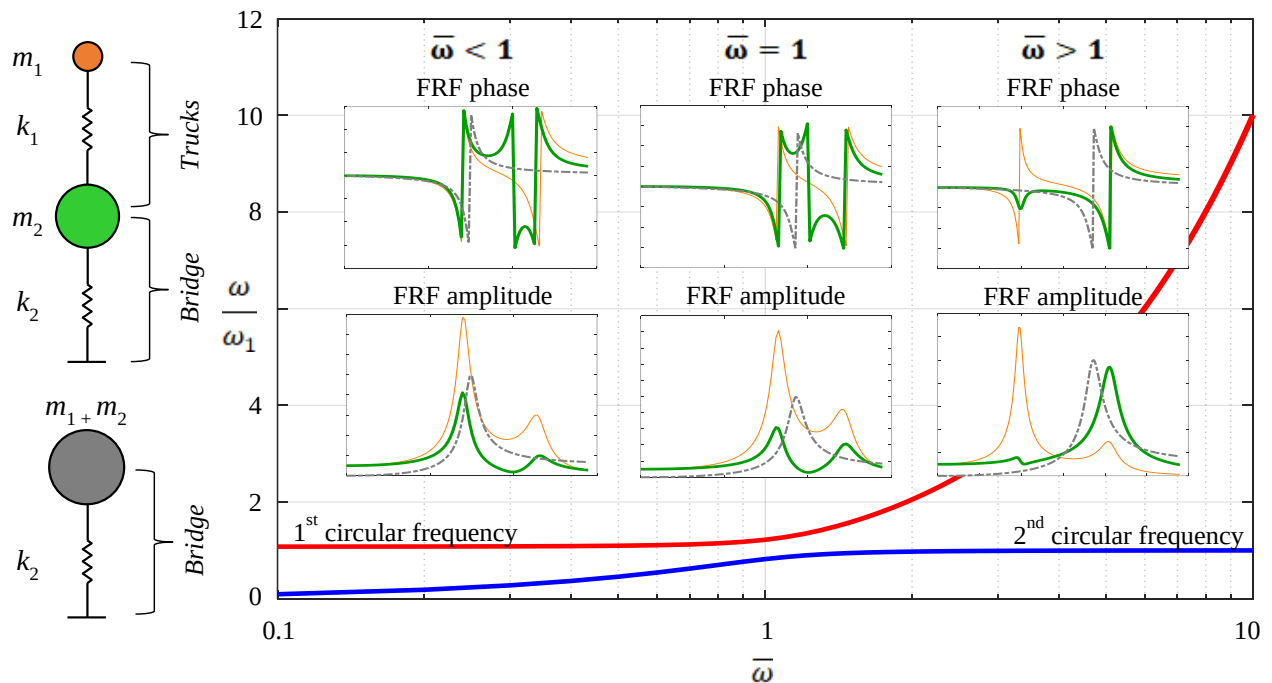


Figure 13. Frequencies and dynamic behaviour of the 2DOF system.

For the sake of simplicity, each truck is modelled with a lumped mass located on the bridge deck in correspondence of the truck centroid. The mass is connected to the deck through elastic links (in the vertical and transverse directions), which are representative of the truck translational stiffnesses. Rotational stiffnesses (relate to bounce, pitch and roll) are neglected as deemed not significant. In the literature some works are available dealing with the dynamic identification of vehicles [41-44]; herein, truck stiffnesses in the vertical and transverse directions are estimated starting from the truck fundamental frequency evaluated from AVTs (through the OMA) performed on a real truck during the static proof test. The fundamental natural frequency identified for the tested full loaded truck is 1.1 Hz. In order to highlight the fundamental frequencies of the bridge transverse and flexural modes, a steady-state analysis is performed on the bridge loaded by configuration C2 by applying unit amplitude distributed forces at the deck level in the transverse and vertical directions, respectively. The FRFs of 2 suitable selected nodes of the deck (namely selected considering the sensor positions and the need to capture first bending and transverse modes) are reported in Fig. 14, comparing functions with the Singular Value Decomposition (SVD) resulting from the OMA procedure. Furthermore, numerical FRFs and SVD relevant to the unloaded configuration U_0 are included to highlight differences between the bridge dynamics subjected to different loading conditions. It is worth mentioning that a perfect matching of peak amplitudes of numerical and experimental curves is not expected since the former consider the response of one node in the vertical and transverse direction while the latter includes the contribution of all the sensors.

Considering the unloaded configuration (U_0), the numerical model is able to capture very well the fundamental frequencies evaluated experimentally. As for the loaded configurations (C1 and C2), numerical values of frequencies identified through peaks in Fig. 14 are compared in Table 8 where $M+K$ refers to the model addressing the truck-bridge interaction while (M) refers to the model only considering the truck masses. The expected duplication of the peaks, due to the truck-bridge interaction, is evident for both the flexural and transverse fundamental frequencies evaluated numerically. Overall, the latter agree quite well with the experimental results and also the relative importance of the peaks is consistent with that estimated through the simple 2DOF system presented in the previous section (Fig. 15) by assuming $\bar{\omega} = 1.07$ and $\bar{\omega} = 1.32$ for the first bending and transverse modes, respectively.

Table 8. Experimental and numerical frequencies for the bridge during P4 – C1 considering the truck-bridge interaction.

Mode n.	Mode name	Frequency [Hz]		
		Experimental	Numerical (M)	Numerical ($M + K$)
1	1 st bending	1.05	1.03	1.05
2	1 st transverse	1.75	1.45	1.68

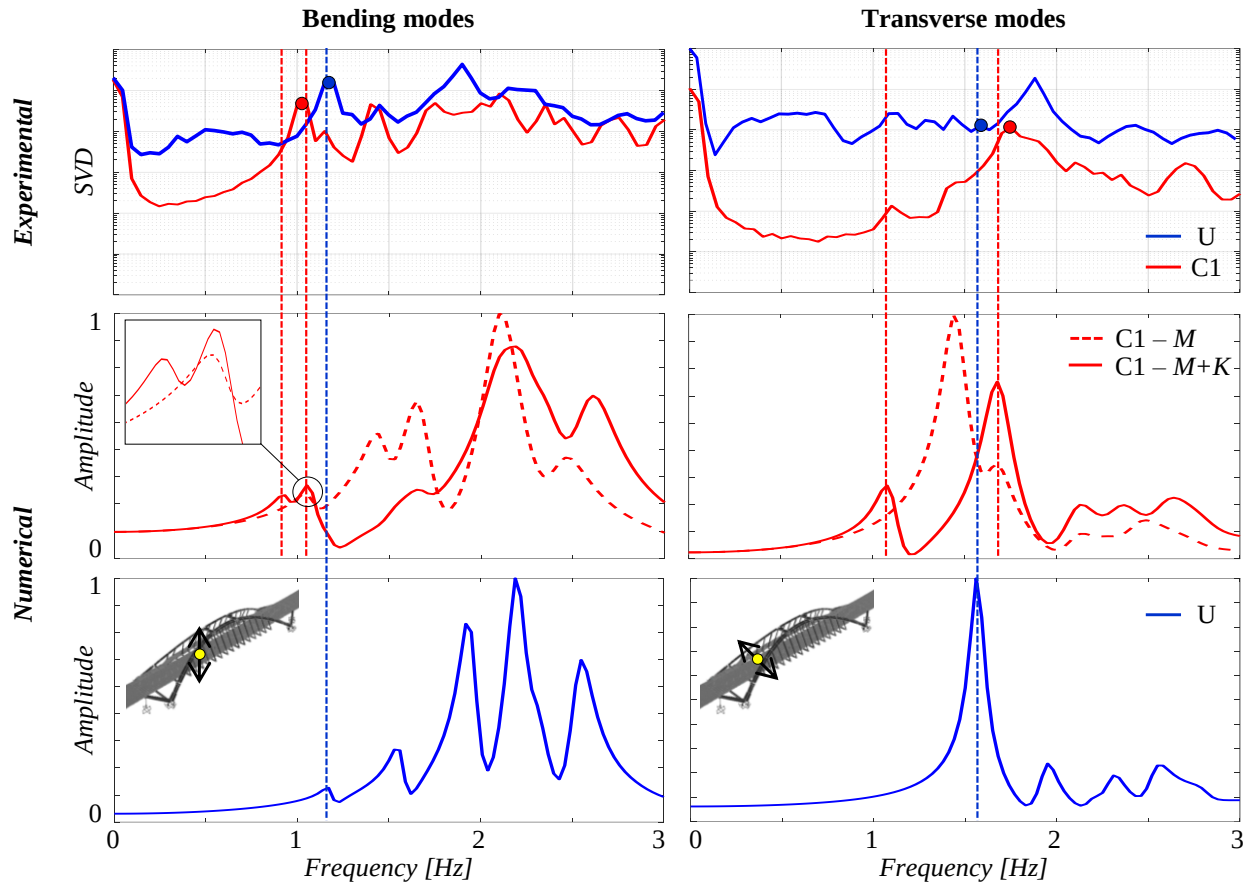


Figure 14. Comparison of experimental tests with numerical FRFs.

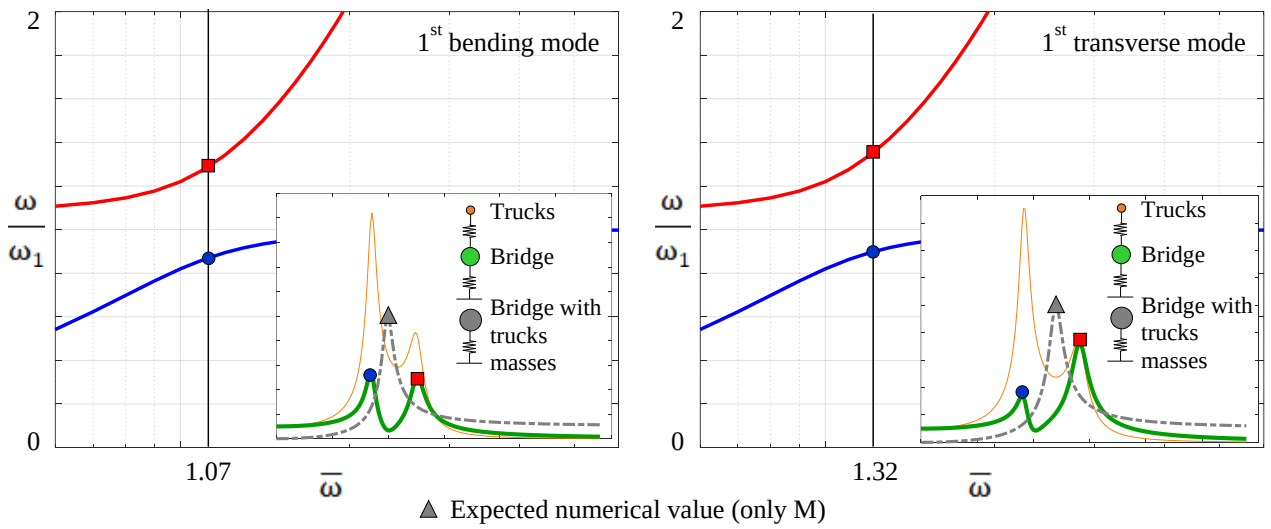


Figure 15. Interpretation of experimental frequencies of the first bending and transverse modes through the 2DOF system.

8 Conclusions

A comprehensive experimental program carried out during the construction phases and the proof static load test of a base-isolated half-through steel arch bridge recently built across the Potenza river has been presented in the paper. Results of impact load tests carried out on hangers and on ties at some deck supports during different construction phases and the final proof test are addressed. Furthermore, during the proof test, relative displacements between the lower and upper plates of the rubber seismic isolators of the arches are measured and the overall dynamic response of the structure is identified through ambient vibration tests, considering different loading conditions.

The experimental program provided useful information during the bridge construction for excluding anomalies in the structural response and for calibrating a reliable numerical model for the interpretation of the final proof test. In detail, fundamental frequencies of hangers and ties measured during the construction phases permitted to control that no anomalous tensile stress states, with respect to design values, develop during the construction. Test results allowed to adjust occasionally the pre-loads in some elements. The dynamic identification of the bridge performed at the end of the construction permitted to evaluate consistency of the numerical predictions obtained through the finite element model of the bridge with the response of the real structure. The model revealed able to capture the bridge dynamics at unloaded conditions very well but failed in capturing the fundamental frequencies measured from ambient vibration tests performed during the proof test. To capture the bridge dynamic response subjected to truck loads during the proof test a more detailed structural model was necessary addressing the truck-bridge interaction problem. It comes out that only modelling truck masses may lead to erroneous prediction of the natural frequencies achieved through tests. In some cases, especially for frequencies of the bridge comparable to that of trucks, neglecting the truck-bridge interaction may lead to underestimate the fundamental frequency of the overall system, leading to a negative evaluation of the dynamic tests outcomes.

Finally, from an overall point of view, results of previous tests, and similar ones that can be performed to assess the compliance of the structure static and dynamic behaviour with the predicted numerical estimations, can be assumed as a key preliminary step for the setup of a structural health monitoring system for the bridge, able to identify possible changes in the bridge performances in time (i.e. by repeating experimental test in time or by monitoring changings in the bridge natural frequencies). In this sense tests have a powerful role in assessing the health status of existing vital infrastructures like bridges.

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