Dynamic response of PC bridge beams under different damages

Life-Cycle of Structures and Infrastructure Systems – Biondini & Frangopol (Eds)

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ABSTRACT: The present paper describes the dynamic test campaign on prestressed concrete bridge beams taken from a dismantled viaduct in Turin, Italy after a service life of 50 years in the framework of BRIDGE|50 research project. Dynamic measurements were previously performed on the decks from which the 29 beams were taken to characterize the behaviour of the viaduct in service condition. Successively the single beams are tested to analyse and evaluate the effects of the different damage levels on the dynamic properties. The vibration data have been collected before the application of static load, after the first cracking condition and after the maximum load applied on the beam to extract the principal modal components. The results highlight the correlation among the evolution of the damage and the dynamic response of the beam and then the effectiveness of vibration tests to identify the occurrence of damages and follow their evolution. The experimental findings could be used in future works to explore the effects of damages of the single beams on the global response of this bridge typology. This work presents the results of the experimental tests on the first eight beams tested.

1 INTRODUCTION

The progressive obsolescence of infrastructures requires to face challenging issues related to the assessment of their residual performances, the increase of the loading conditions, the extension of their service life and how to monitor the evolution of the health state over the years. Nevertheless, the effectiveness of the monitoring strategies requires to deepen the knowledge of the way the combination of different factors, such as corrosion or deterioration, extreme weather events, increase of loads, material ageing, that may influence the structural reliability.

The BRIDGE|50 research project, started in 2020, aims at deeply investigate the behaviour of aged structural elements approaching the end of their service lives (Biondini et al. 2021). The research activities will be devoted to accurately investigate the structural performances and characterize the material properties of a representative portion of a viaduct dismantled after 50 years of service life, namely 25 prestressed concrete beams, 4 box girders and 2 pier caps. The experimental campaign should represent a great source of information to be used in developing predictive models and calibrating monitoring techniques.

Vibration-based methods are widely applied in structural health monitoring techniques to follow the evolution of structural integrity, due to the direct dependence between the dynamic behaviour and the structural stiffness. Moreover, their appeal has increased due to the ease of managing the continuous streaming of large amount of data and inferring information by means of data mining algorithms (Demarie, Sabia, 2019).

In literature, several studies report the results of experimental dynamic investigations on beam-like prestressed structures, focusing on their use as symptoms of the onset of damage or the evolution of deterioration over time forces (Quattrone et al. 2012, Guiglia et al. 2014). In several cases, the authors have pointed out the low sensitivity of modal parameters to identify the reduction of prestressing. For this reason, tests under controlled conditions on end-of-life structures can contribute to supply more accurate definition of sensitivity limits and the thresholds to be transferred to monitoring systems.

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The girders of Bridge|50 project were tested both in their current conditions and after the induction of defined damage steps. The aim is to estimate the contribution of the single beams on the global response of the decks, previously tested before the dismantling, and to investigate the influence of damages of the single beams on the global behaviour.

In this paper the results of the dynamic test campaign on a set of five beams are reported. The beams have the same nominal geometrical and material characteristics but some different damage. The dynamic data have been collected before the execution of the static tests, in cracked state and at the end of static tests, after the application of the ultimate loading condition.

2 THE CORSO GROSSETO VIADUCT

The Corso Grosseto viaduct was built in 1970. It was originally constituted by 80 concrete decks having variable spans, from 16 m to 24 m, and a total length of 1400 m (Figure 1). The structural elements which are investigating in BRIDGE|50 research project belong to the first four decks of the southern part of the viaduct. Each deck was composed of 10 precast prestressed concrete I-beams and 2 external precast prestressed concrete U-beams, simply supported at piers, and completed by two transverse beams at the thirds of the span and the 0.14 m thick upper slab were both casted on site. The dimensions of the I-beams cross section and the on-site casted slab, object of the present work, are shown in Figure 2. A set of 25 prestressed concrete beams constituting the decks have been dismounted and collected in a testing area, where a bench test has been realized to perform both static and dynamic tests.

Figure 1. An historical view of the Corso Grosseto viaduct.

3 STATIC TEST CAMPAIGN

According to the preliminary visual inspection carried out to map the presence of corrosion, cracks, and spalling, the beams have been classified in good structural condition (Table 1). The beams B07 and B08 have been damaged during the dismantling phases, when strands at the bottom flanges were cut. The B07 had two strands cut on both sides of the lower flange, at distances 3.7, 9.6, and 15.5 m from the end section. The B08 girder had two strands cut on one side at distances of 3.8 and 15.5 m.

The static test procedure consists in applying an incremental static load up to the reaching the ultimate load capacity by means of four hydraulic jacks. An intermediate unloading phase was made to evaluate the residual prestressing load and the effects of the opening of the cracks on the dynamic parameters (Savino et al. 2023). The test on B02 beam has been

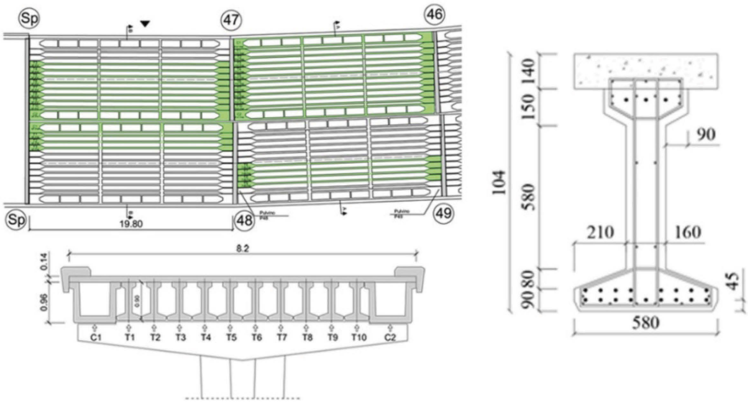

Figure 2. Drawing of the first four decks of the southern part of the viaduct. In green, the beams collected to be tested. On left, the I-shaped beam cross section (measures in mm).

performed in a three-point bending scheme (shear span 9.3 m), whereas a four-point scheme (shear span 6.5 m) was adopted for testing B05 to B08 aimed at investigating the maximum flexural capacity and the shear-bending interaction, respectively. The beams have been loaded up to reach the ultimate condition highlighting a brittle rupture due to the crushing of the upper slab. The principal results are summarized in Table 1. The ultimate flexural strengths M_R take into account the load applied by the hydraulic jacks and the weight of the steel elements used to apply the load to the tested girder.

Table 1. Results of the static tests.

Beam	Position on the deck	Test configuration	M_R (kN m)	Condition
B02	B8 – P47/46	Three Point	1675	Undamaged
B05	B6 – P47/46	Four Point	1656	Undamaged
B06	B6 – Ab/P47	Four Point	1720	Undamaged
B07	B6 – P48/47	Four Point	1445	Damaged (4 strands cut)
B08	B10 – Ab/P47	Four Point	1684	Damaged (2 strands cut)

4 DYNAMIC TEST CAMPAIGN

The description of the test setup is reported in Figure 3. The setup is made of 10 capacitive 1 V/g accelerometers and an instrumented hammer.

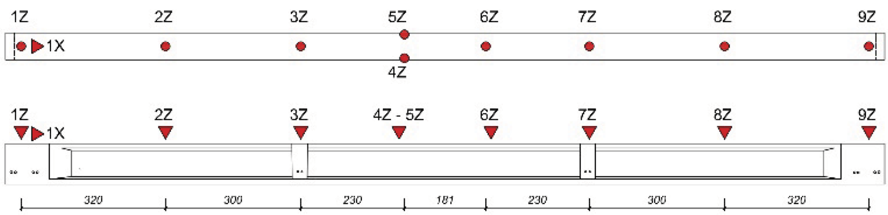

Figure 3. Dynamic test setup.

The accelerometers have been placed on the upper slab along the longitudinal axis of the beam, spaced to get an adequate spatial resolution of the vertical flexural modes. The sensors 1X on the rolling support and the couple of accelerometers 4Z and 5Z placed near the midspan on the edges are used to discriminate the longitudinal and torsional dynamic components.

The acceleration signals were acquired at a 512 Hz sampling frequency for both ambient noise and impact hammer excitations. The impact tests were carried out by hitting the beams with an instrumented hammer in vertical direction in the halfway of two consecutive accelerometers. To replicate as close as possible the onsite supporting conditions of the deck, the bench components lying on the upper slab during the static tests are each time removed.

The dynamic test campaigns have been performed before the static test (Phase 0), after the intermediate unloading cycle (Phase 1) and after the application of the ultimate load (Phase 2).

4.1 *Experimental modal analysis*

The extraction of the principal modal components is performed evaluating the frequency response function (FRF) matrix with respect of the input given by means of the instrumented hammer. When dealing with experimental or operational modal analysis, the FRF is defined for a linear system by the relationship between the forcing function $\{F(\omega)\}$ and the p structural responses $\{Y(\omega)\}$, usually recorded by accelerometers when measured during in-situ testing. In the frequency domain the FRF matrix $[H(\omega)]$ in described by equation (1):

$$\underset{(p\ x\ 1)}{\{Y(\omega)\}} = \underset{(p\ x\ q)}{[H(\omega)]}\,\underset{(q\ x\ 1)}{\{F(\omega)\}} \tag{1}$$

The element of FRF matrix $H_{k,j}(\omega)$ maps the input $F_k(\omega)$, applied at the k-th degree of freedom (DOF), and the output Y_j measured at the j-th DOF. In case of acceleration as output signals, the FRF matrix is expressed in the accelerance form (Ljung, 1999). The generic element of the accelerance matrix, expressed in terms of modal properties, is given by:

$$H_{j,k}(\omega) = \frac{\ddot{Y}_j(\omega)}{F_k(\omega)} = -\omega^2 \sum\nolimits_{r=1}^{N} \frac{1}{m_r} \frac{{}^r\psi_j{}^r\psi_k}{\omega_r^2 - \omega^2 + 2i\omega\omega_r} \tag{2}$$

The FRF can be estimated by classical H_1 or H_2 estimators, given by the ratio between the cross-spectrum $S_{fy}(\omega)$ of the output $Y(\omega)$ and the input $F_k(\omega)$ and the auto-spectrum of the input $S_{ff}(\omega)$ or of the output $S_{yy}(\omega)$ that can be chosen according to whether the major source of noise is for input or output signals. The FRF is a complex function which amplitude has a maximum at the resonant frequency. The magnitude of the modal coefficient is simply taken as the value of the imaginary part at resonance. The sign is either positive or negative, considering the direction of the peak along imaginary axis. This implies that the phase angle is either 0° or 180°.

Figure 4 shows an example of FRF, magnitude and imaginary part, from the excitation and the time history measured at the position from 1Z to 9Z on the beam.

After a preliminary analysis of the spectral content, the signals were pre-treated by filtering out of the bandwidth [0 - 100 Hz], removing the trends. The FRFs have been estimated basing on each singular impulse acceleration data and then averaged for each hitting position. The first three modal components are clearly identified as shown by the example proposed in Figure 4. The mode shapes associated to these components correspond to the three main flexural modes of the beam. The modal coordinates are estimated by the imaginary part of the FRFs.

5 RESULTS OF THE DYNAMIC IDENTIFICATION

In this paragraph the result of the dynamic identification for each of the five beams for each of the three phases are reported. The tests phases are depicted in Figure 5. The Phase 0 aims at investigating the dynamic properties of the structural elements before the execution of static test, describing the condition after 50 years of service life. The results of the dynamic tests campaign performed on the decks from which the beams have been removed are reported in (Quattrone et al., 2021).

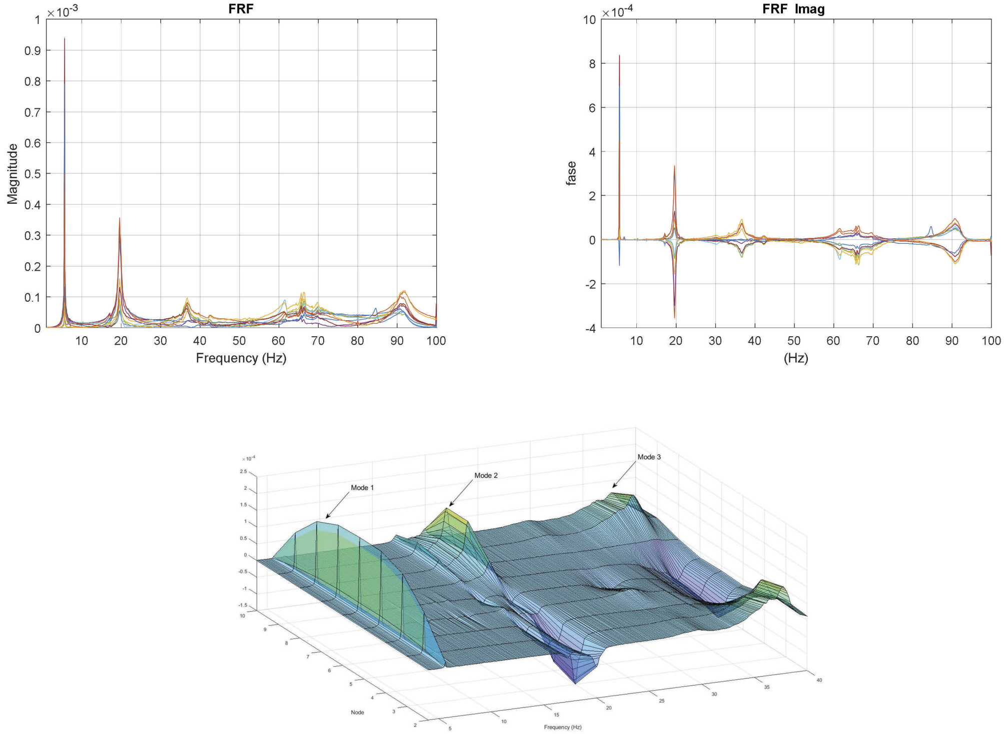

Figure 4. Example of FRF. Left: Magnitude; Right: Imaginary part. Bottom: Mode Shapes.

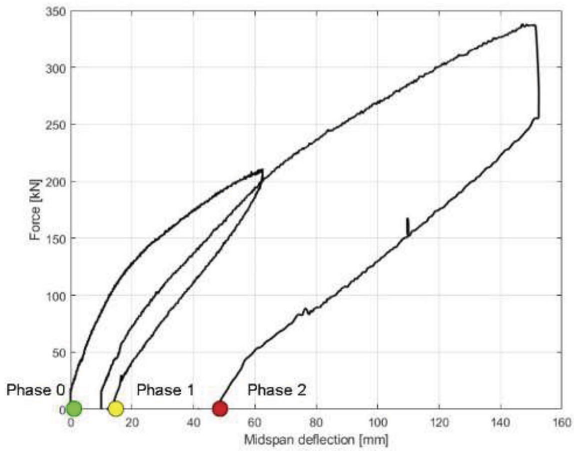

Figure 5. Force – midspan deflection. The dots indicate the phase test.

The Phase 1 corresponds to the unloading condition, subsequent the reaching of the full cracked condition, identified in Table 2 as a fraction of the ultimate bending moment reached during the static tests. The Phase 2 regards the dynamic test on the beams which experienced their ultimate load condition.

Table 2. Damage levels corresponding to Phase 1 tests.

Specimen	M (kNm)	% M_R
B02	1080.7	68.7%
B05	884.0	55.8%
B06	939.9	57.0%
B07	855.7	62.3%
B08	910.7	56.5%

The Table 3 reports the mean values of frequencies and damping estimated at the three phases for each beam.

Table 3. Mean modal frequencies and damping ratios.

		Phase 0			Phase 1			Phase 2		
		mode 1	mode 2	mode 3	mode 1	mode 2	mode 3	mode 1	mode 2	mode 3
B02	f (Hz)	5.49	19.38	36.44	5.28	18.88	36.34	4.78	18.58	32.69
	ξ(%)	0.37	0.84	2.18	0.48	0.81	2.19	0.55	1.00	2.22
B05	f (Hz)	5.47	19.52	36.62	5.31	19.30	35.50	5.05	19.45	33.44
	ξ(%)	0.34	0.80	1.94	0.40	0.74	1.68	0.95	2.16	2.03
B06	f (Hz)	5.60	19.86	37.21	5.47	19.48	36.17	5.01	18.50	33.62
	ξ(%)	0.36	1.30	0.85	0.40	1.33	0.91	0.63	1.06	1.57
B07	f (Hz)	5.58	19.83	35.22	5.48	19.36	34.91	5.01	19.09	33.84
	ξ(%)	0.39	0.65	1.77	0.39	0.68	1.99	0.42	1.30	2.14
B08	f (Hz)	5.61	20.35	38.25	5.41	19.58	37.08	4.83	18.33	33.85
	ξ(%)	0.29	1.46	1.32	0.31	1.04	1.57	0.43	1.06	1.57

The following figures illustrate, as an example, the comparison between the mode shapes at different damage levels (phases 0, 1 and 2) for girder B08. It worth noting that small but noticeable differences can be observed with the increasing of damages. Similar results have been found for the other girders.

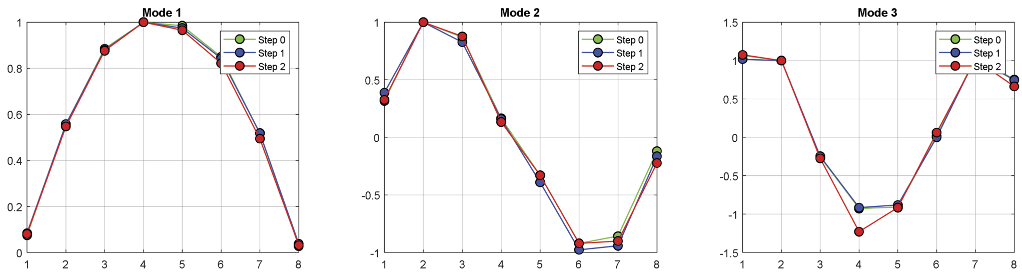

Figure 6. Beam B08 mode shapes at different damage levels.

The Figure 6 shows the distributions of frequencies and damping identified at the different damage levels. It can be noted that at the increase of the damage level correspond the reduction of the modal frequencies and the increasing trend of damping ratios. Moreover, the scattering of both the parameters increase.

The reductions of the frequencies of the first mode vary in range of about 2-4% for Phase 1 tests and about 8-14% for Phase 2. The entities of these reductions are comparable to the findings reported in similar experimental campaign present in literature (Quattrone et al. 2012).

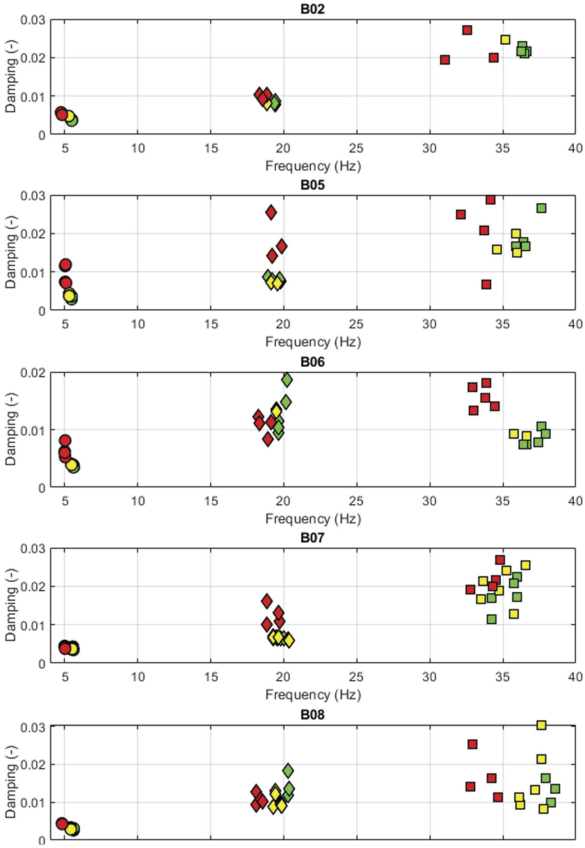

Figure 7. Distribution of frequencies and damping ratios. The colors correspond to the phases as indicated in Figure 4.

6 CONCLUSIONS

This work presents the results of the experimental dynamic campaign performed on a set of bridge girders dismounted after 50 years of service life. The tests are part of the BRIDGE|50 research project.

The girders have been characterized for different damage levels induced by static tests. The variation of the modal parameters have been observed and the results confirm the low sensitivity of modal parameters to the damages for prestressed concrete structures. Albeit the small magnitude, the variations of the structural condition are clearly detectable, confirming the capability of the dynamic structural identification to follow the evolution of damage. The experimental results here presented should be very useful in the field of permanent structural health monitoring. Future works will refine the estimation of damping ratio and the implementation of damage localization and quantification techniques.

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Nuova Economia; ATI Itinera & C.M.B.; ATI Despe & Perino Piero; Quaranta Group. BRIDGE|50 website: http://www.bridge50.org.

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