# Dynamic tests and modal identification of Corso Grosseto viaduct decks before the dismounting

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ABSTRACT: This paper describes the dynamic test campaign carried out on four prestressed concrete decks being part of the viaduct of Corso Grosseto in Turin, before its demolition after a service life of 50 years. Subsequently, 25 beams, 4 box girders and 2 pier caps were collected from the tested decks to be deeply investigated within the frame of BRIDGE|50 research project. The purpose of the preliminary test campaign is to characterize the dynamic behavior of the decks in their service condition. Dynamic measurements were acquired on each deck by using different excitation sources. The collected data have been analyzed in order to extract the principal modal components and the outcomes have been compared to the analytical results given by a FE model. The results of the tests will be used to better investigate the effectiveness of the dynamic tests in the assessment of ageing infrastructures, to explore the effects of damages on single beams and its influence on the global response of this bridge typology.

## 1 INTRODUCTION

Nowadays, the examples of issues associated with the obsolescence of infrastructures are constantly increasing (Dunker & Rabbat 1993, Zheng et al. 2015). Recent tragic events have spotlighted the urgency of a constant assessment of the safety condition of numerous large structures. Authorities responsible for their management and maintenance are committed to the difficult task of quantifying the efforts needed both in terms of economic resources and qualified competencies (Frangopol and Liu 2007).

Causes and effects of structural deterioration and incorporation of damage models in life-cycle reliability analysis methods have been widely investigated (Fernando 2015, Šomodíková 2016, Biondini & Frangopol 2016, 2019). However, little research effort has focused on damage identification and validation of numerical models based on full-scale experimental testing on existing critical bridges (Quattrone et al. 2012). Indeed, structural condition assessment is strictly connected with Structural Health Monitoring (SHM), which aims to evaluate the in-service behavior of a structure along its service life (Aktan et al. 2001, Brownjohn 2006). Both the research subjects suffer the impossibility to systematically perform large testing campaigns in controlled environment to reproduce the whole life-cycle of a full-scale structure. This complicates the widespread diffusion of these techniques in on-site applications except for strategic assets, subsequently limiting the development of standard procedures that inform optimal maintenance strategies.

In this context, the BRIDGE|50 research campaign (Biondini et al. 2020) will represent an invaluable opportunity to deeply investigate the behavior of aged structural elements approaching the end of their service lives. The research activities will be devoted to testing a representative portion of the whole viaduct before its dismantling after 50 years of service life, namely 25 prestressed concrete beams, 4 box girders and 2 pier caps. Each element will be deeply investigated in its current state and then progressively damaged up to collapse. It may also allow to test the effectiveness of retrofitting intervention and to validate health monitoring and condition assessment procedures.

The purpose of the experimental campaign described in this paper is to carry out dynamic testing on the last standing decks before the dismantling. Although additional damage may have been induced by the ongoing demolition works, the tests allow to characterize the global dynamic behavior of the bridge in its actual operational condition. The results will be used to evaluate the impact of deterioration damage on the residual load carrying capacity of the bridge.

## 2 DYNAMIC TESTING

The fast-growing diffusion of sensors with limited cost and dimensions and the rise of new information data technologies, such as 5G and IoT, is fostering the integration of different monitoring strategies within the condition assessment process of infrastructures. Currently, vibration testing is one of the most used techniques for condition assessment of large structures, such us highway bridges, due to its capability to provide information about the structural performance measuring the mechanical response in operational conditions. Measured data allow to extract the dynamic properties, i.e. modal parameters (natural frequencies, mode shapes and modal damping values), and infer about the global behavior due to the correlation between structural vibration and several mechanical parameters, such as stiffness, mass distribution, constraints effectiveness, presence of damage patterns and deterioration phenomena.

### 2.1 *Corso Grosseto viaduct*

The Corso Grosseto viaduct was built in 1970. It was originally constituted by 81 concrete decks having variable spans, from 16 m to 24 m, and a total length of 1400 m (Figure 1). The structural elements which will be investigated in the BRIDGE|50 research project belong to the first four decks of the southern part of the viaduct highlighted in Figure 1 by the red box within the plan view.

All the four decks were built adopting the same structural configuration (Figure 2) with a simply supported static scheme. Each deck comprises ten I-shape pre-cast prestressed concrete beams and two edge precast pre-stressed box beams. Two transverse beams at the thirds of the span and the 0.14 m thick upper slab were both casted on site. At the extremities, the beams are transversally connected by post-tensioning rebars and lean on rubber supports placed above the prestressed pier caps and the abutments. The decks are completed by the concrete bases of the safety barriers, which have no structural function. In the investigated portion of the bridge, the unique central barrier rests on the adjoining decks. At the time of testing, the pavement was already removed.

### 2.2 *Experimental setup*

The decks were tested one at a time, adopting the same sensor layout depicted in Figure 3. A set of 22 sensors is used, 21 of them placed on the investigated deck and one kept on the midspan of the adjacent one. The sensors are described as follows:

- 6 uniaxial 10 V/g piezoelectric accelerometers PCB 393B31 (Figure 3, yellow circles)
- 10 uniaxial 1 V/g capacitive accelerometers PCB 3701G3FA3G (Figure 3, purple diamonds)
- 6 triaxial 1 V/g piezoelectric accelerometers PCB 356B18 (Figure 3, green squares).

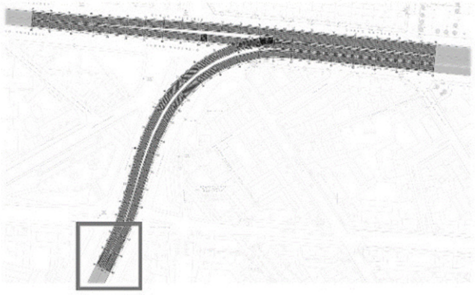

Figure 1. Top: a view of the investigated decks. Bottom: plan view of Corso Grosseto viaduct. The red box indicates the investigated decks.

The accelerometers have been glued on the upper slab and the vibrations have been simultaneously acquired at a sampling frequency of 256 Hz by a 24-bit data acquisition system. The setup is mainly designed to get a dense spatial resolution in vertical direction, which are acquired in all the 21 positions indicated in Figure 3. The most sensitive accelerometers, having a sensitivity of 10 V/g, are placed at the extremities to get vibrations possibly influenced by pier caps or movements of the bearings. The lateral and transversal vibrations are measured in five locations as visible in Figure 3 with squared green bullets. Finally, a triaxial accelerometer is placed at the center of the adjacent deck to investigate its dynamic interaction.

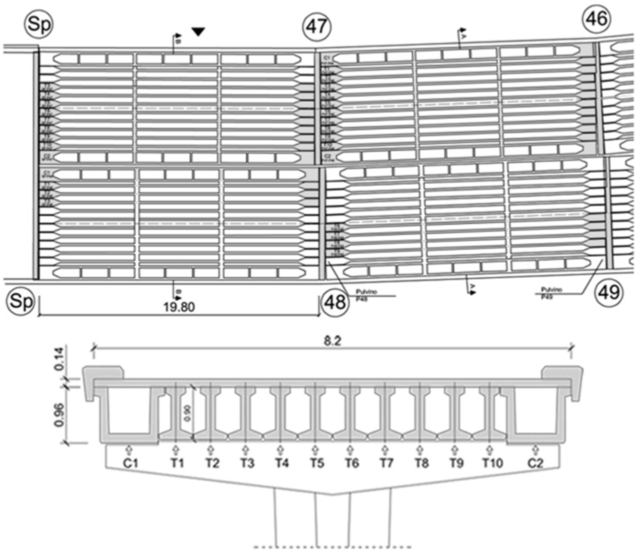

Figure 2. Drawings of the four decks subjected to dynamic tests and the section of the deck. In grey, the beams dismounted that will be tested during BRIDGE|50 research.

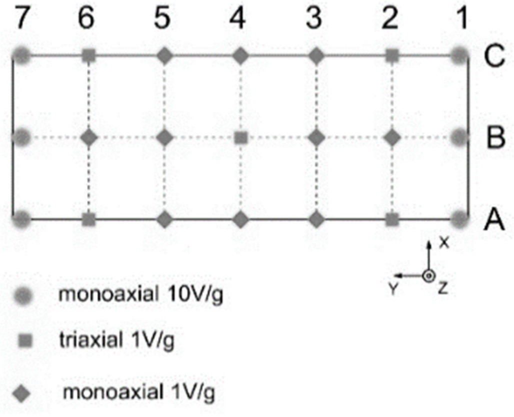

Figure 3. Dynamic test setup. Position of the sensors (left) and the assigned labeling of the decks (right).

Each deck, identified according to Figure 4, has been evaluated through a sequence of measurements under different conditions:

- Impacts by an instrumented hammer (IH);
- Impacts by a falling mass (IM);
- Ambiental vibration (AV).

The hitting positions of the hammer and free-falling mass are illustrated in Figure 5. The impacts were applied by using a hammer, having a mass of 5.5 kg, with a force transducer mounted on its head to record the impulsive force applied. The data have been collected hitting the slab in vertical direction in five different positions, chosen in order to excite both flexural and torsional modes. Several impacts were applied at each position, spaced in time intervals large enough to run out the effect of the previous stroke.

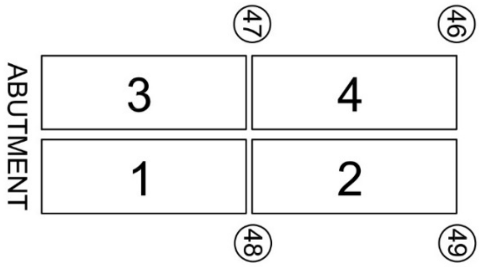

Figure 4. Dynamic test setup. The assigned labeling of the decks.

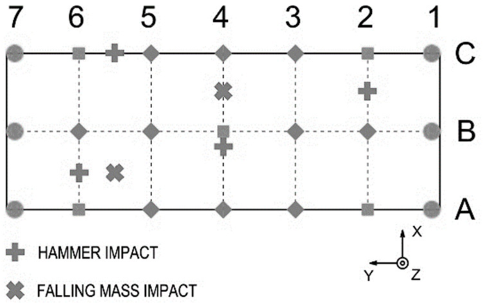

Figure 5. Impact testing. Red crosses indicate the hammering positions, the blue X indicate the mass falling positions.

In order to apply a higher impulsive force, the impact tests have been also applied using a free-falling mass. The system is constituted by a steel mass suspended at a known height by a wire (Figure 6). In a first trial, made on Deck 2, a total mass of 200 kg falling from 0.5 m and 1 m was explored. The other decks were tested in same positions reducing the mass twofold in order to avoid the overloading of the closer accelerometers.

Finally, ambient vibrations have been acquired for at least 30 minutes, avoiding external action (i.e. moving people) acting directly on the deck.

Figure 6. The free-falling mass device (200 kg configuration).

### 2.3 *Data analysis and modal identification*

The acquired signals have been pre-processed to roughly define the energy content interval in frequency domain and then conditioning by subsampling, mean removal and detrending through a polynomial fitting. The preliminary analysis of the spectra showed that the mutual influence of consecutive decks can be neglected (Figure 7).

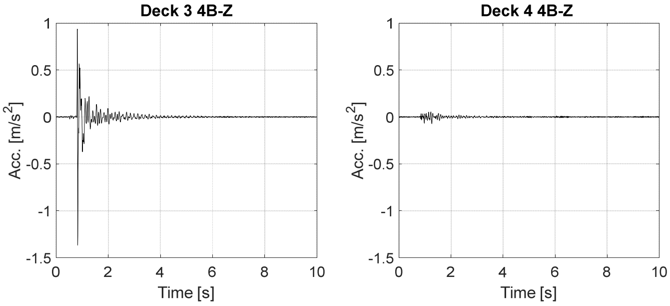

Figure 7. Signals recorded during a mass impact test (Deck 3, mass 100 kg, position 4, between B and C) in vertical directions in points 4B of Deck 3 and 4B of Deck 4.

Only the vertical acceleration signals were used since the decks exhibited no significant lateral vibrations. Two different time domain techniques were used to identify the modal parameters from impact testing and ambient vibration recordings.

In IH and IM tests, the recording were split to comprise a singular impact and the full decay of the amplitude of vibrations. The free-decay responses were subsequently analyzed using the Eigenvalue Realization Algorithm (ERA). The AV tests were split into a sequence of 180 s recordings, with a superimposition lag of 20 s, and the modal parameters repeatedly estimated adopting the Stochastic Subspace Identification (CVA-SSI) algorithm. Both the adopted identification techniques are broadly described in literature and a rigorous discussion of their formulation is out of the scope of this paper. A detailed description of ERA algorithm can be found in Juang & Pappa (1985), whereas the CVA-SSI is described in Peeters & De Roeck (1999).

For each typology of testing, all the modal parameters have been collected and sifted applying a series of tolerance criteria to discard computational modes. A preliminary selection is made discarding modes with damping $\zeta > 10\%$ and frequencies lying out of the interval (4, 30) Hz. The remaining modes have been grouped basing on natural frequencies, damping values and the relative resemblance of their modal shapes evaluated through the Modal Assurance Criterion (MAC). The chosen parameters are:

- Frequency variation < 5 %;
- Damping variation < 20 %;
- MAC > 95 %.

Table 1 reports the mean values of the modal frequencies and damping identified for each test typology. A similar modal shape sequence is found for all the decks and all the test typologies. For instance, the Figure 8 represents the modal shapes identified from AV tests on Deck 1.

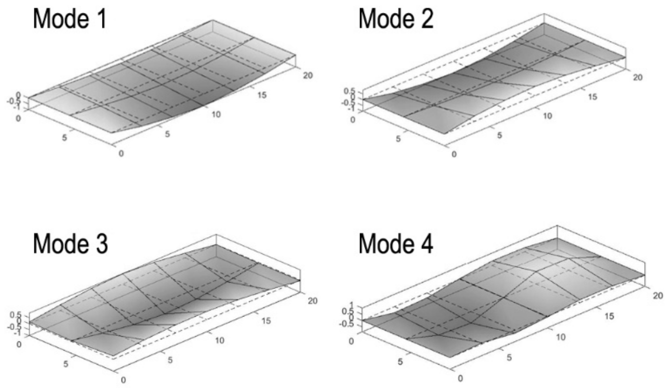

Figure 8. Deck 1. First four experimental modal shapes.

Table 1. Experimental modal frequencies and damping.

| | Mode # | Deck 1 | | Deck 2 | | Deck 3 | | Deck 4 | |
|---|---|---|---|---|---|---|---|---|---|
| | | $f$ [Hz] | $\zeta$ [-] | $f$ [Hz] | $\zeta$ [-] | $f$ [Hz] | $\zeta$ [-] | $f$ [Hz] | $\zeta$ [-] |
| IH | 1 | 6.64 | 0.027 | 5.99 | 0.017 | 6.97 | 0.028 | 6.17 | 0.038 |
| | 2 | 8.24 | 0.013 | 7.17 | 0.013 | 8.26 | 0.008 | 7.18 | 0.039 |
| | 3 | 14.10 | 0.012 | 13.30 | 0.009 | 14.01 | 0.009 | 12.95 | 0.010 |
| | 4 | 24.30 | 0.012 | 19.89 | 0.024 | n.d. | n.d. | 19.05 | 0.035 |
| IM | 1 | 6.51 | 0.028 | 5.92 | 0.019 | 6.97 | 0.028 | 6.08 | 0.034 |
| | 2 | 8.15 | 0.012 | 7.11 | 0.017 | 8.24 | 0.007 | 7.07 | 0.018 |
| | 3 | 13.82 | 0.014 | 12.99 | 0.016 | 14.07 | 0.011 | 12.63 | 0.006 |
| | 4 | 20.80 | 0.013 | 19.29 | 0.042 | 20.82 | 0.039 | 18.50 | 0.038 |
| AV | 1 | 6.68 | 0.017 | 5.97 | 0.012 | 6.85 | 0.027 | 6.08 | 0.008 |
| | 2 | 8.26 | 0.011 | 7.23 | 0.017 | 8.30 | 0.027 | 7.10 | 0.017 |
| | 3 | 14.23 | 0.006 | n.d. | n.d. | 14.12 | 0.005 | 13.10 | 0.037 |
| | 4 | 21.63 | 0.009 | 18.42 | 0.036 | 21.60 | 0.023 | 18.65 | 0.018 |

## 3 FINITE ELEMENT MODELLING

The interpretation of the experimental data often benefits from the use of analytical models. In order to better interpret the experimental results, a preliminary finite element (FE) model of the typical deck is realized according to onsite surveys and original drawings. The FE model is constituted by 14146 nodes and 8984 solid elements having linear, elastic and isotropic material properties.

The model is built according to the following initial hypotheses. The prestressed precast beams are modeled as effectively connected by the transversal beams, whereas the contribution of the transversal prestressed rods at the terminal parts is not modeled. The safety barriers are modeled as a mass of 500 kg/m distributed along the external upper nodes of the slab and null stiffness.

The elastic moduli of precast beams and cast-in-place elements, the slab and the transverse beams, are differentiated in order to take into consideration their expected different properties. The deck is modeled as a "pinned-roller" supported structure, assuming efficient expansion joints and bearings, negligible influence of piers and pier caps and restrained transversal displacements at the extremities.

Under the initial hypotheses, good agreement is found in terms of the first two modal frequencies and shapes (Figure 9 and Table 2). Figure 9 shows a good qualitative agreement between analytical and experimental modal shapes. It is worth noting the first modal shape highlights a transversal asymmetry, founded in all the decks, which cause a higher deflection in correspondence of the external lateral border. It seems indicating that the presence of the central concrete barrier lightly influences the dynamics of adjoined decks, imposing an asymmetric lateral boundary condition. This contribution will be taken into account in future works by a suitable model updating procedure.

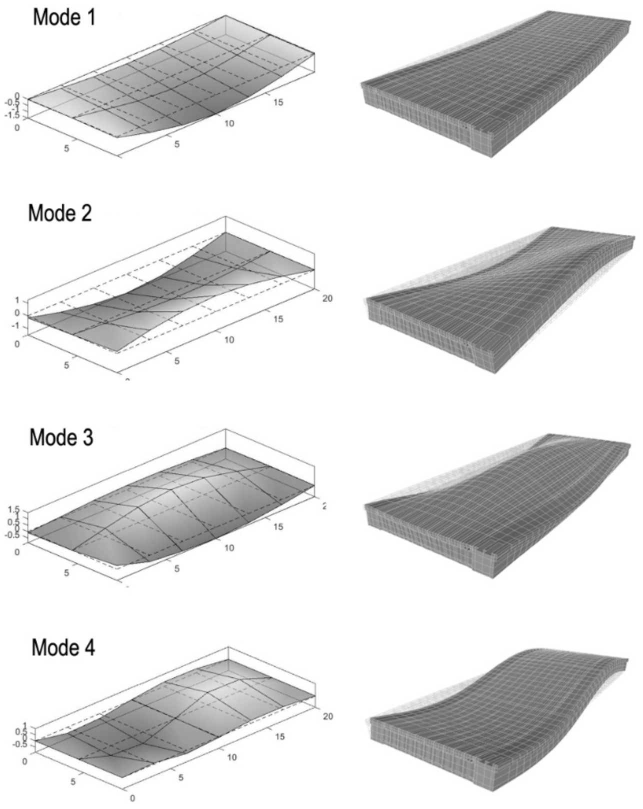

Figure 9. Comparison between experimental and analytical modal shapes.

The most significant difference is observed comparing the experimental mode n.3, highly influenced by the transversal rigidity of the deck.

Table 2. Comparison between analytical and experimental modal frequencies (AV tests).

| FEM | | | | Experimental | |
|---|---|---|---|---|---|
| Mode # | Preliminary Model 1 [Hz] | Model 2 [Hz] | Model 3 [Hz] | Deck 1-3 [Hz] | Deck 2-4 [Hz] |
| 1 | 6.04 | 5.99 | 7.70 | 6.75 | 6.02 |
| 2 | 8.61 | 7.73 | 8.27 | 8.28 | 7.17 |
| 3 | 26.94(*) | 15.32 | 16.47 | 14.18 | 13.10 |
| 4 | 20.18 | 20.16 | 20.31 | 21.62 | 18.65 |

(*) 6th mode in FE model

In the FE modal analysis, it appears in a different order, the sixth instead of the fourth, having a frequency of about twice than the experimental one. This suggests that the initial assumptions lead to overestimating the effectiveness of the transversal connection between the beams. A rough calibration was done to reduce the error of the analytical modes by progressively decreasing the stiffness of the cast-in-place transverse beam. The result of the procedure indicates that the transversal connection between the beams can be assumed as almost ineffective. This can also be acknowledged by the pictures in Figure 10, showing two beams after the cut required to move the beams in the testing site location, confirming the poor quality of the transversal beams concrete casting and the lack of continuity with the longitudinal precast beams.

Figure 10. Photos of the cast-in-place transversal beams of two precast beams after the dismantling.

Analysing the experimental frequencies reported in Table 1, it is possible to observe a noticeable difference between the frequencies of Decks 1 and 3, lying on southern side on the abutment, which are 11% higher with respect the ones of the Decks 2 and 4, lying on the piers. This may be associated with a possible different behaviour of the bearings. In order to explore this hypothesis, the FE boundary conditions were modified adopting a pinned bearing at the southern end. Table 2 compares the mean experimental frequencies computed for the pairs Decks 1 & 3 and Decks 2 & 4, and the analytical ones calculated by three FE models, whose characteristics are summarized below:

- Model 1: preliminary model having "Pinned – roller" restrains and the contribution of the transversal beams fully effective;
- Model 2: "Pinned – roller" restrains; transversal beams ineffective;
- Model 3: "Pinned – pinned" restrains; transversal beams ineffective.

Several causes can lead to this different behaviour, such as a poor effectiveness of the joints between deck and abutment as well as the contribution of the deformability of the pier and pier cups, which is not modelled.

A full model updating procedure will help reducing the gap in frequencies, detecting the proper stiffness distribution in transversal direction and the effects of the mutual influence between the adjoint decks. Moreover, it will be possible to investigate intermediate constrains conditions which appear more likely to occur in real cases.

## 4 CONCLUSIONS AND FUTURE WORKS

This paper describes the dynamic test campaign performed on the four decks belonging to Corso Grosseto viaduct before the dismantling. The tests performed had the main purpose of characterizing the global behaviour of the decks in their operational conditions before the dismantling. The main modal parameters have been identified for the four decks and used to calibrate a representative FE model. The combined use of experimental results and numerical modeling allowed to highlight the lack of transversal stiffness of the deck and the role of bearings in the global structural behaviour.

A refined FE model of the deck properly updated based on the dynamic tests herein described would represent an effective tool to evaluate the influence of different typologies of damage, introducing the data collected from the experimental test on the single beams. In this context, the research project BRIDGE|50 will study the mechanical properties and residual structural capacity of the beams collected from the tested decks. The results of experimental responses of the beams, affected by the deterioration experienced along the 50 years of service life as well as subjected to controlled damage patterns, will be introduced in the FE model to evaluate the contribution of different detrimental aspects on the global response of the decks.

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