



Comparison Between Model Prediction and Measured Response of a Prestressed Concrete Bridge Tested to Failure

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Abstract. The number of bridges that are approaching or exceeding their initial design life has increased radically. Meanwhile, an ever-increasing volume of traffic each year, both in number and weight of vehicles, is creating an additional critical situation for this kind of structure. To predict the response of bridges to traffic loads and their ultimate capacity with low uncertainties, we can use numerical structural models; however, such uncertainties increase as bridges age due to deterioration mechanisms. Non-destructive tests of material specimens and full-scale on-site load tests of the structure allow to update model parameters and have a better estimate of the bridge behaviour. However, different load tests provide different information with different impacts on the updated model accuracy. With the aid of a real-life case study, the Alveo Vecchio highway bridge, which has been tested to failure with a sequence of progressively increasing load, we aim to understand what behaviour can predict a structural model and what we can learn from a load test. This study is part of a research agreement between the Italian Ministry of Sustainable Infrastructure and Mobility, Autostrade per l'Italia SpA (the main operator of Italian highways), and the University of Trento. It concerns the management and monitoring of civil infrastructure intending to develop survey protocols and monitor systems to assess the safety and performance of existing highway bridges.

Keywords: Load test · Bridge behavior · Numerical structural models · Ultimate capacity · Model updating

1 Introduction

Most of the European bridges have been determined to be over 40 years old [1]. While Belgium and Sweden count more than 50% of their bridges to be constructed in years that predate 1980; Germany, Denmark, Finland, and the Netherlands stand out with a percentage of bridges over 40 years old which goes beyond 70% [2].

We estimated that in Italy, 50.4% of the highway bridges are at least 50 years old and the average age of Italian bridges is 48 years. At the same time, bridges are withstanding an ever-increasing volume of traffic each year, both in the number and the weight of vehicles. The European Standards [3] and the Italian Standard [4] have progressively increased the design traffic load over the years [5] and the actual dimensional and mass limits imposed by national’s Highway Codes [6, 7] confirm such an increase. Considering the investments required to maintain the performance and safety of bridges in operation, as well as the impact of bridges failure on the economy, environment, and society [8, 9], it is crucial to frequently check bridge’s deterioration, prioritize the maintenance in an infrastructure asset, and effectively assess the performance and safety condition of bridges [10, 11]. Non-destructive tests (NDTs) [12], structural health monitoring (SHM) [13, 14], and diagnostic and proof load tests [15] can help civil infrastructure managers monitor the properties of material and the response of bridges in operational conditions and under extreme loads. Numerical models can predict the response of bridges to traffic loads and their ultimate capacity with low uncertainties; however, such uncertainties increase as bridges age due to deterioration mechanisms [16]. In this paper, we aim to investigate the correlation between the model prediction and the observed response of a prestressed concrete highway We predicted its structural response with two analytical and two numerical models, with a progressive level of refinement and model parameters defined firstly a priori and then a posteriori of NDTs and monitoring data.

2 Alveo Vecchio Viaduct and Load Test Protocol

The Alveo Vecchio viaduct was built in 1968 and decommissioned in 2005 after a landslide hit which cause the displacement of one of its piers. It is representative of 52% of the Italian highway bridges in terms of structural type, age, and deterioration state. The prestressing was applied through 14 post-tensioned parabolic cables per girder. Figure 1 shows a lateral view and a cross-section of the viaduct. Technical information about the structure is found in [17].

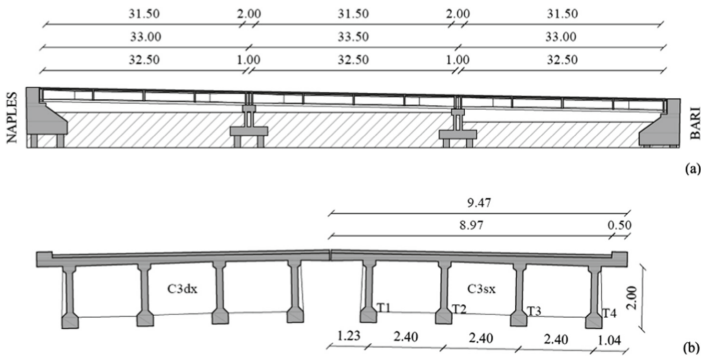


Fig. 1. (a) Lateral view and (b) cross section of the Alveo Vecchio viaduct.

A visual inspection reported that the landslide did not affect spans C3sx and C3dx. Therefore, span C3sx was chosen for the load test to failure reported in this paper.

As a load unit, we chose the concentrated load in the midspan that generates a bending moment in the most loaded girder equal to the design bending moment: 4259 kNm. As a result, the load unit was chosen as 2400 kN and realized with a matrix of 3×4 steel ballast weights of 100 kN each, arranged in two layers. To collapse the viaduct, we decided to increase progressively the number of weights layers in multiples of the load units. The load test protocol, schematized in Fig. 2, consisted of 5 loading phases, each of them followed by the complete unloading of the bridge: P1 – 1200 kN, P2 – 2400 kN, P3 – 4800 kN, P4 – 7200 kN, P5 – 9600 kN.

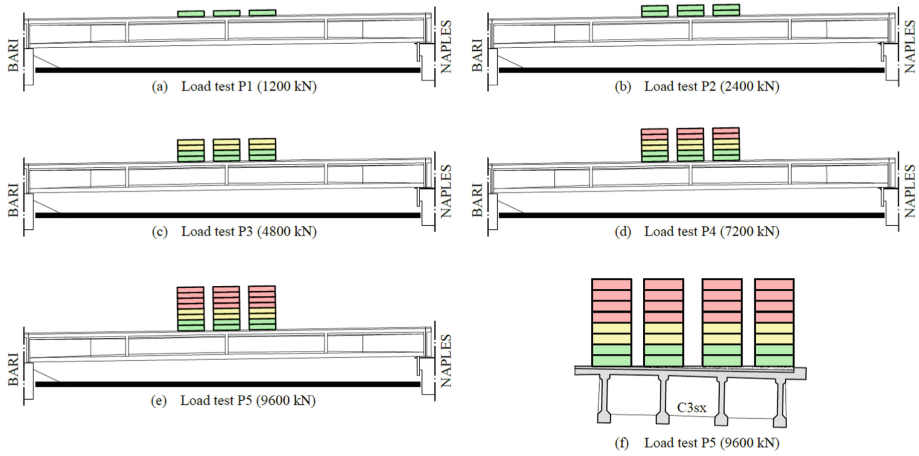


Fig. 2. Loading phases P1–P5. Colors: green is 1 times the load unit; yellow is 2 times the load unit; red is 3 and 4 times the load unit.

3 Prediction Models

To predict the viaduct response during the load test, we developed four structural models and we have progressively updated them based on the NDTs on material specimens and the measurements for a monitoring system. We developed two analytic models (ANA1 and ANA2) and two based on finite elements (FEM1 and FEM2). We defined three hypotheses on the mechanical properties of materials based on the distributions of the results from an extensive campaign of non-destructive tests (NDTs), and we used them as input for the prediction models. We assumed as Gaussian the distributions of results from NDTs. The three hypotheses are summarized in Table 1.

Table 1. Hypotheses on materials properties based on NDTs, (F = fractal, PS = prestressing).

Location	Material	Property	Unit	Hp A (F 5%)	Hp B (F 50%)	Hp C (F 95%)
Slab	Concrete	Compressive cylinder strength	N/mm ²	27.3	31.9	36.4
Slab	Concrete	Elastic modulus	N/mm ²	29323	31942	34561
Girder	Concrete	Compressive cylinder strength	N/mm ²	33.5	41.5	49.5
Girder	Concrete	Tensile strength	N/mm ²	0	2.18	2.45
Girder	Concrete	Elastic modulus	N/mm ²	30774	33523	36272
Girder	PS steel	Yield stress	N/mm ²	1385	1509	1633
Girder	PS steel	Ultimate tensile strength	N/mm ²	1485	1618	1751
Girder	PS steel	Strain at maximum load	%	3.5	4.43	6.06
Girder	PS steel	Residual stress	N/mm ²	482	619	1000
Girder	Steel	Yield stress	N/mm ²	269	399	529
Girder	Steel	Ultimate tensile strength	N/mm ²	534	614	694

Model ANA1. In the analytical models ANA1 and ANA2, we assumed the following parameters as deterministic: the ultimate strain of concrete $\epsilon_{cu} = 0.35\%$ and the elastic modulus of the prestress steel $E_s = 200$ GPa. We assumed the parabola-rectangle diagram for concrete under compression from EN 1992-1-1:2004 [18] and the bilinear diagram for harmonic and reinforcing steel as stress-strain relation curves of materials. The model ANA1 we implemented is based only on materials properties a priori and it was used to define the stop-criteria of the load test.

Model ANA2. According to the results obtained from the loading phase P3, we updated the residual stress and removed hypothesis A since it seemed excessively precautionary. Table 2 shows the updated value of residual stress $\sigma_{p,\infty}$ for each girder.

Table 2. Updated value of residual stress and elastic modulus of concrete in model ANA2.

Hypothesis	Residual stress $\sigma_{p,\infty}$ [MPa]			
	T1	T2	T3	T4
B	1030	975	980	1005
C	955	980	1020	950

Model FEM1. We developed the finite element model FEM1 with the open-source software OpenSees [19]. We used the material hypothesis C defined for model ANA2 and we modelled each girder with frame elements with fiber sections to permit the spread of plasticity, better simulate the geometry of girders and the variation of the position of

the prestressing cables along the girder. We assigned to each fiber the following non-linear stress-strain relations already implemented in OpenSees: “Concrete 04 - Popovics Concrete Material” for the concrete, “Steel04 Material” for the harmonic steel, and “Steel02 Material – Giuffr.- Menegotto-Pinto Model with Isotropic Strain Hardening” for the steel reinforcement.

Model FEM2. We updated model FEM1 into model FEM2 by changing the residual stress of prestressing cables $\sigma_{p,\infty}$ and the elastic modulus E_c of the concrete in each girder. Based on a sensitivity analysis, we chose the values that allow us to better fit the observed response to the load test measured by the monitoring system. Table 3 shows the updated value of $\sigma_{p,\infty}$ and E_c for each girder.

Table 3. Updated value of residual stress and elastic modulus of concrete in model FEM2.

Parameter	T1	T2	T3	T4
Residual stress $\sigma_{p,\infty}$ [MPa]	696	794	820	950
Elastic modulus E_c [MPa]	39899	39899	39899	39899

4 Monitoring System

We designed the monitoring system based on the Italian Guidelines for SHM, UNI/TR 11634:2016 [20]. The study presented in this manuscript focuses mainly on deflection and crack-opening at the bottom of the girders. We measured such quantities with 34 PT1DC_2/5/20 Celesco and 22 PZ12-100/A-25, respectively; their accuracies are 0.3 mm for the first and $2.5 \cdot 10^{-3}$ mm for the latter. Figure 3 shows the layout of the entire SHM system, which consists of 119 sensors divided into 8 types: (a) wire displacement sensors for the deflection of girders, rectilinear displacement transducers for the crushing of bearings and the settlement of pier and abutment, and biaxial tiltmeters for the inclination of the pier, abutment, and girders; (b) rectilinear displacement transducers for cracks opening and top slab deformation, and acoustic emission sensors; and (c) platinum resistance thermometers for temperature and accelerometers for dynamic monitoring.

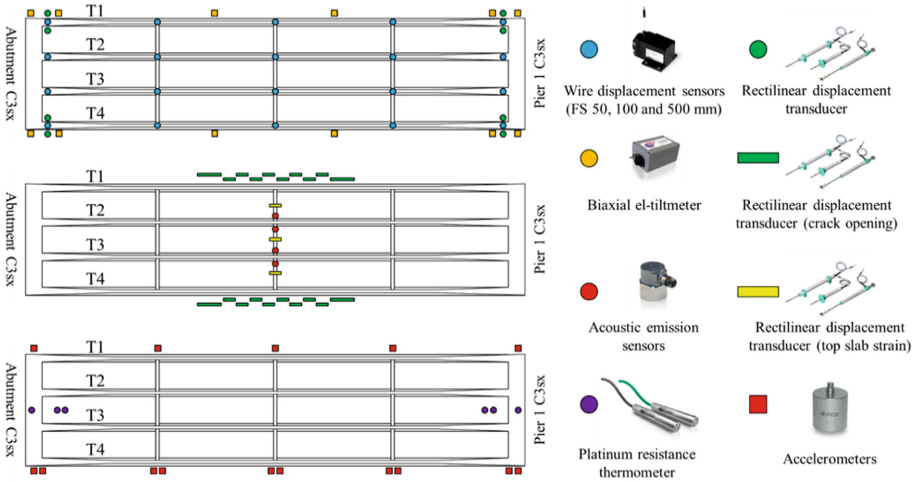


Fig. 3. Layout of the SHM system installed on the Alveo Vecchio viaduct during the load test

5 Load Test and Results

A preliminary load test on the pier and abutment of span C3sx allowed us to verify that the 2005 landslide did not alter their capacity. Then, on 4th July 2019, the flexural load test started as planned in the load test protocol; it continued until 23rd July 2019, when it ended at 9300 kN during the loading phase P5 due to the occurrence of the stop criterion: a total deflection of the girder T1 higher than 300 mm. Figure 4 shows a picture of the bridge loaded at the end of the last loading phase P5.



Fig. 4. Pictures of the Alveo Vecchio viaduct during the P5 phase

For the objective of this paper, the most important results quantities measured during the load tests are deflections of girders and crack openings. The first is particularly significant because (i) we defined the stop criteria in terms of deflection measurements, and (ii) we aim to compare the deflection measured during the test with those predicted by the structural models; the second is because deflections are strongly affected by the initiation and propagation of cracks in concrete. Figure 5 shows the deflections measured during the loading phases P1–P5, Fig. 6 shows the crack pattern on the girder T4 at the end of the load test. The girder T1 experienced a greater deflection than the others, up

to 30% more, probably because of the higher stiffness of the girder T4, the influence of the cross-girders in the redistribution of the loads, and the different initiation and propagation of concrete cracks that progressively and differently changed the stiffness of each girder. Indeed, the girder T1 experienced a significant change in the strain at 3300 kN during the load phase P3, which marks the opening of the first crack and a change in its structural response from stage I (elastic) to state II (cracked). In contrast, the first crack opened at 4000 kN in the girder T4.

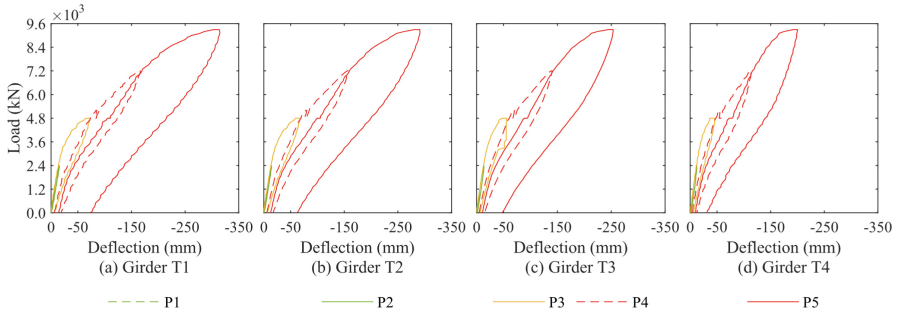


Fig. 5. Deflection of the midspan of the girders during the loading phases P1–P5.



Fig. 6. Visible cracks in the middle of the girder T4 opened during the loading phases P3 (amber triangles), P4 (single red triangles), and P5 (double red triangles).

6 Discussion

Figure 7 reports the predicted and observed vertical deflection of the girders. Based on the comparison between ANA1’s predictions and the measured response we can observe that this analytical model, based only on hypotheses defined a priori of the load test, predicted only the elastic response of the bridge in the uncracked state (loading phase P2). Then, based on the comparison between ANA2’s predictions and the measured response, we observe that this model, with the residual stress updated based on the results of the load test, can predict both the elastic stiffness and the first-crack load of the bridge in the uncracked state (loading phases P2 and P3). However, ANA2 cannot

predict the response of the bridge when it has pre-existent cracks (loading phase P4). Moving on to FE models, based on the comparison between FEM1's predictions and the measured response, what we observe is similar to what was observed with model ANA2: the elastic stiffness and the ultimate capacity predicted are close to the observed ones. However, the first crack load is different, because we used as input of FEM1 the residual stress updated to make the model ANA2 fit the measured response better than model ANA1; thus, the residual stress needed to be updated again. Finally, based on the comparison between FEM2's predictions and the measured response we observed that this model fitted better the response of the bridge measured during the entire load test.

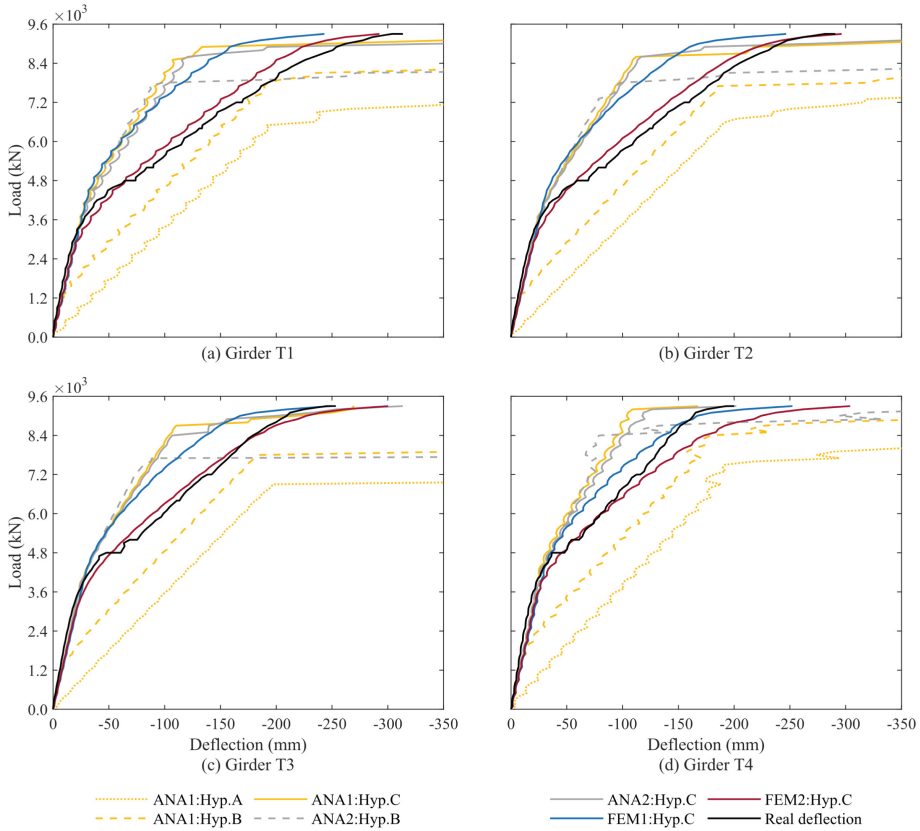


Fig. 7. Comparison between the predicted and the observed deflections of the girders

Those results confirm that different load tests (load tests up to the design load of uncracked and cracked bridges, higher than the design load, and up to the failure of the bridge) provide different information with different impacts on the prediction of the updated model. The load tests performed up to the design traffic load (P2, P3, P4 up to 2400 kN) allowed us to observe the elastic stiffness and to discriminate whether the viaduct had pre-existent concrete cracks. They also pointed out whether the opening of the first crack happens during the test (P3) or not (P2), and the first-crack load. In

contrast, they did not provide any useful information about the ultimate capacity. On the other hand, the load tests up to the collapse of the bridge (P5), pointed out that the ultimate capacity depended mainly on the tensile and compressive strength of the concrete, the yield and ultimate tensile strength of prestressing cables' steel, and the ultimate strain of concrete and steel. However, all prediction models with hypothesis C on the properties of materials (fractal 95% of the distribution of results obtained from NDTs of material specimens), provided a good estimation of the ultimate load-carry capacity, with an error $<5\%$. Therefore, to better estimate the ultimate capacity, it seems more important to perform NDTs rather than on-site load tests of bridges.

7 Conclusions

Numerical models can predict the response of bridges to traffic loads and their ultimate capacity with low uncertainties; however, such uncertainties increase as bridges age due to deterioration mechanisms. In this paper, we reported the structural response of a PC bridge to load test carried on until its failure performed in five phases with progressively increasing loads. We defined both an analytical and a numerical FE model to predict the bridge's response, which we updated progressively based on NDTs of material specimens and monitoring system results acquired during the test. Based on the predicted and the observed results, we discussed what information the load test provided, what behavior those structural models predicted, which are the model parameters that influence the prediction the most, and whether the predictions improved thanks to a model updating. Our results point out that: the elastic modulus of concrete was the parameter that influenced the elastic stiffness the most, the residual stress of prestressing cable influenced the first-crack load and the crack stiffness the most, and the ultimate capacity seemed to depend mainly on the tensile and compressive strength of the concrete, the yield and ultimate tensile strength of prestressing cables' steel, and the ultimate strain of concrete and steel. Regarding the prediction models, both analytical and FE models based on design documentation and NDTs gave a prediction of the elastic stiffness of the girders with an error $<20\%$ and of the ultimate capacity with an error $<5\%$ if the properties of materials have been carefully estimated through NDTs. Moreover, the load test up to the design traffic load allowed recognizing whether the PC bridge had pre-existent concrete cracks or not by comparing the observed stiffness with the predicted one. It also pointed out whether the opening of the first crack happened during the test. On the other hand, when the PC bridge had no pre-existent concrete cracks, the load test up to the design traffic load did not provide useful information to estimate the residual stress of prestressing cables and the bridge's ultimate capacity. To do so, the load test must crack the girders, which is unsuitable for in-service bridges. In this case, it is advisable to perform NDTs of material specimens to estimate the residual stress. Finally, we observed that a load test carried on up to the failure of a bridge can be particularly effective when it is part of an infrastructure stock that consists of many bridges with similar structural type, age, and deterioration state, like the Alveo Vecchio viaduct. Indeed, the load test results allowed us to verify that the load-carry capacity of this bridge was almost four times the design load. Most of the Italian bridges with similar features likely have similar ultimate capacity.

Acknowledgments. This research has been supported by the Italian Ministry of Sustainable Infrastructures and Mobility (MIMS) and Autostrade per l'Italia SpA (ASPI), MIUR PON RI 2014–2020 Program (project MITIGO, ARS01_00964), Consorzio ReLUIIS (Project ReLUIIS Ponti 2021–2022 ‘Implementation of provisions of DM 578/2020’) and Fondazione CARITRO Cassa di Risparmio di Trento e Rovereto (grant number 2021.0224).

References

1. BELL, Brian; RAIL, Network. European railway bridge demography. E FP, 2004, 6
2. Del Grosso, A., Inaudi, D., Pardi, L.: Overview of European activities in the health monitoring of bridges. In: First International Conference on Bridge Maintenance, Safety and Management, Barcelona (2002)
3. European Committee for Standardization, «EN 1992-2 Eurocode 2 - Design of concrete structures - Concrete bridges - Design and detailing rules» CEN, Brussels (2005)
4. Italian Ministry of Infrastructure and Transport. Norme Tecniche per le Costruzioni D.M. 17/01/2018 (2018)
5. Bencivenga, P., et al.: Evolution of design traffic loads for Italian road bridges. In: Pellegrino, C., Faleschini, F., Zanini, M.A., Matos, J.C., Casas, J.R., Strauss, A. (eds.) Proceedings of the 1st Conference of the European Association on Quality Control of Bridges and Structures, vol. 200, pp. 1351–1358. Springer, Cham (2022). https://doi.org/10.1007/978-3-030-91877-4_154
6. UK Department for Transport, Driving Standards Agency, The Official Highway Code 2007 Edition, The Stationery Office (2017)
7. Italian Ministry of Infrastructure and Transport. Codice della strada (2021). <https://www.aci.it/i-servizi/normative/codice-della-strada.html>
8. Frangopol, D.M.: Maintenance and management of civil infrastructure based on condition, safety, optimization, and life-cycle cost. *Struct. Infrastruct. Eng.* **3**(1), 29–41 (2007)
9. Verzobio, A., Bolognani, D., Quigley, J., Zonta, D.: Quantifying the benefit of structural health monitoring: can the value of information be negative? *Struct. Infrastruct. Eng.* **18**, 573–594 (2021)
10. Caspani, V.F., Tonelli, D., Poli, F., Zonta, D.: Designing a structural health monitoring system accounting for temperature compensation. *Infrastructures* **7**(1), 5 (2022)
11. Verzobio, A., Bolognani, D., Quigley, J., Zonta, D.: Consequences of representativeness bias on SHM-based decision-making. *Struct. Infrastruct. Eng.* **18**, 851–863 (2021)
12. Cotini, O., Di Maggio, R., Tonelli, D., Nascimben, R., Ataollahi, N.: Porosity of a fast-setting mortar with crystallization admixture and effect of a SA-PA modification. *Materials* **15**(4), 1542 (2022)
13. Tonelli, D., et al.: Effectiveness of acoustic emission monitoring for in-service prestressed concrete bridges. In: Proceedings of SPIE (2021)
14. Quqa, S., Giordano, P.F., Limongelli, M.P., Landi, L., Diotallevi, P.P.: Clump interpolation error for the identification of damage on board decentralized sensing systems. *Smart Struct. Syst.* **27**(2), 351–363 (2021)
15. Lantsoght, E.O.L., van der Veen, C., de Boer, A., Hordijk, D.A.: State-of-the-art on load testing of concrete bridges. *Eng. Struct.* **150**, 231–241 (2017)
16. Abbiati, G., Covi, P., Tondini, N., Bursi, O.S., Stojadinović, B.: A real-time hybrid fire simulation method based on dynamic relaxation and partitioned time integration. *J. Eng. Mech.* **146**(91) (2020)
17. SPEA. Relazione di Calcolo Viadotto Alveo Vecchio alla progr. 39 + 364.21, Opera N°14 (1966)

18. European Committee for Standardization, EN 1992-1-1 Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings, CEN, Brussels (2005)
19. Pacific Earthquake Engineering Research. OpenSees (2006). <https://opensees.berkeley.edu/>. Accessed 2021
20. Italian National Unification, UNI/TR 11634:2016 Guidelines for structural health monitoring, UNI, Rome, (2016)