Large-scale experimental testing of 50-year-old prestressed concrete bridge girder

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ABSTRACT: This paper reports on large-scale loading tests survey carried out on 50-year-old prestressed concrete girders as part of the BRIDGE|50 research project (www.bridge50.org). The girders were retrieved from an existing viaduct in Turin, Italy. The prestressed concrete elements were 19.2 m long and had an I-shaped cross-section with a cast-in-situ slab. Variable damages were found, due to in-service deterioration and/or subsequent lifting operations during the demolition phase. Each specimen was subjected to static tests using monotonic or cyclic loading up to the ultimate load. This paper reports the results of tests on the second group of four girders, comparing their load-deflection responses and strains with those of the first group subjected to a different static scheme. The experimental findings highlight the global structural response from bending to bending/shear failure; the outcomes will define a valuable reference database for assessing the residual structural performance of existing girder bridges.

1 INTRODUCTION

Civil infrastructures operate under adverse service conditions, which may cause durability problems at different levels and, in the worst cases, the decrease or total loss of load-carrying capacity. Meanwhile, traffic demand is continuously growing, both in terms of number and weight of vehicles. However, as civil infrastructures age, uncertainties on the prediction of structural response increase due to the deterioration mechanisms. As a result, structural assessment has received much attention in recent decades from different perspectives. Structural health monitoring technologies have been developed to report the structural health status and early warning in presence of structural damage. Indeed, full-scale load tests are conducted to better understand the structural behavior of existing structures and to calibrate structural models. However, large-scale tests on civil infrastructures, are not always straightforward. In order the learn more about non-linear behavior and the ultimate capacity of bridges, the load test involves irreversible damages which led to the dismissing of the structure. Furthermore, huge human, financial, and material efforts are required, and as reported in Recupero & Spinella (2019), only few studies have been carried out on full-scale prestressed concrete (PC) girders from decommissioned bridges around the world (Wang et al. 2020). Assessment of existing bridges requires a thorough understanding of their structural behavior. However, the investigation usually concerns few structural members, equipped to study mainly the flexural behavior. Full-scale experiments should include more variables to be controlled. A systematic assessment of several specimens should be conducted under different loading test setups for a better understanding of the failure mechanism and thus develop more reliable estimates of the residual capacity.

In this scenario, the BRIDGE|50 research project was established to study the residual structural performance of a 50-year-old viaduct and provide a framework for safety assessment and residual lifetime evaluation of existing bridges (Biondini et al. 2020, 2021). A wide experimental campaign was planned to investigate 25 prestressed concrete (PC) I-girders, 4 PC box girders, and 2 pier caps recovered from the C.so Grosseto viaduct, a viaduct built in Turin in 1970 and

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decommissioned in 2018. After demolition, the structural members were stored in a testing site properly equipped to conduct a detailed structural assessment including visual inspection, deterioration mapping, and non-destructive tests (Anghileri et al. 2020). Furthermore, a large reaction steel frame of the Interdepartmental Center for the Safety of Infrastructures and Constructions (SISCON) of Politecnico di Torino was used to perform full-scale load tests up to failure. Several load test setups were considered to study the failure mechanisms under flexural, mixed flexural/shear, and shear loading modes. The first group of four PC beams was tested under three-point bending configuration and detailed in Tondolo et al. (2021, 2022) and Savino et al. (2023). This paper reports the experimental results of the second group of four beams tested under four-point bending configuration up to failure. The reported parameters not only extend the database aimed at providing further knowledge on the structural response of 50-year-old PC beams but could be used to evaluate developed theories on resisting mechanism. The observed response is also compared with the structural behavior recorded for the first group of beams.

2 DESCRIPTION OF THE C.SO GROSSETO VIADUCT

The C.so Grosseto viaduct was a multilevel road bridge located in Turin, Italy. It was built in 1970 and decommissioned in 2019 for a new urban redevelopment plan (Savino et al. 2020). Its structural topology represents a typical solution widespread in the Italian motorway heritage and consists of two structurally independent decks, one for each traffic way, with simply supported span. Each span comprised ten PC I-shaped girders and two PC box girders at the edges, with a length averaging from 16.0 to 24.0 m. Two diaphragms connected the longitudinal girders at the third points. The superstructure was completed by a cast-in-situ slab of 140 mm. The infrastructure was 1.4 km long and achieved a maximum vertical clearance of 12.4 m.

During the decommissioning works in 2018, a total of 31 structural elements were removed and stored in a testing site of about 6000 m^2, located in Turin. A detailed layout of the preserved structural member in the original deck configuration is shown in Figure 1. The coloured elements represent the girders recovered for BRIDGE|50 research project; the elements depicted in blue belong to the first group of tests whereas the elements highlighted in red refer to the girders investigated and reported in this work. All the elements were properly identified with a code, according to the original configuration. The PC beams studied in the present work were identified as B7-P47/46, B6-Ab/P47, B6-P48/49, and B10-Ab/P47 according to classification reported in Savino et al. 2023.

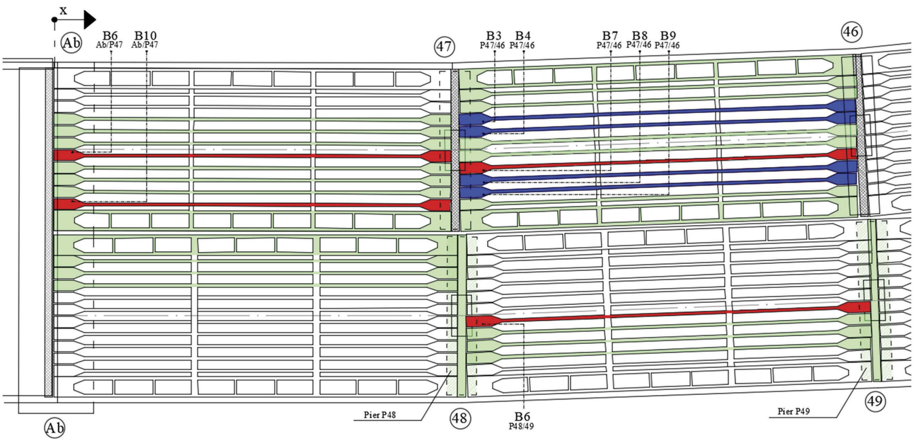


Figure 1. In-service layout of the investigated structural members (first group of tested girders in blue, second group of tested girders in red).

2.1 *Details of the tested girders*

All the girders were 19.5 m long, with an I-shaped cross-section of 580 x 900 mm. Cover depths were 45 mm for prestressed strands and were variable up to a minimum of 10 mm for ordinary reinforcement. According to the design documentation, prestressing reinforcement was composed of seven-wire strands with a nominal diameter of 12.7 mm and tensile strength of 1638 MPa. The prestressed reinforcement consisted of 17 strands concentrated in the lower part of the beam and 3 strands located in the top flange as pictured in Figure 2. The stirrups are made by 8 mm ribbed steel spaced of 250 mm. The 14 cm cast-in-situ slab was reinforced with a square mesh of rebars. The maximum allowable stress for the strands at the tensioning stage was 1400 MPa. The concrete compressive strength at 28 days was designed to be 30 MPa. No specific requirements were reported for the cast-in-situ slab. The arrangement of the reinforcement as well as the cross-section geometry is given in Figure 2.

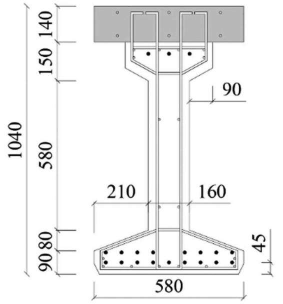


Figure 2. I-beam cross-section (units in mm).

According to the diagnostic campaign performed before the load tests session, the beams investigated in the present work were considered in good condition except for B6-P48/49 and B10-Ab/P47 girder which presented controlled damages induced by dismantling operations (Savino at al. 2023). The B6-P49/48 beam 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 B10-Ab/P47 girder had two strands cut on one side at distances of 3.8 and 15.5 m. Relying on the results provided by the preliminary diagnostic campaign, the load test program was properly planned, starting from the undamaged beams, considered as a benchmark for the subsequent beams with different degrees of damage.

3 FULL-SCALE LOAD TESTS SETUP

For the purpose of testing the PC girders with variable load configurations, a proper reaction steel frame was used (SISCON). Each beam was simply supported and loaded under a four-point bending configuration adopting shear spans of 650 cm. The loading system consisted of two couple of hydraulic jacks which transfer the load to the specimens through transverse steel beams. The loading process was conducted in two loading cycles. In the first phase, the load was increased until the opening of cracks, followed by the complete unloading of the girder. In the second loading phase, the specimens were loaded up to failure. The load tests were performed with a fixed loading rate by controlling the force and were stopped when the concrete in the compression zone crushed. The double loading cycles allowed also to perform the dynamic characterization of the specimens at different level of damages (Quattrone et al. 2020, Sabia et al. 2021, 2022).

As it is unusual to have full-scale elements of existing bridges loaded to collapse, the data gathered during such tests are of extreme interest to understand their structural response. For this reason, a proper measurement plan has been designed for each beam to measure several parameters involving displacements, strains, loads, vibrations, and acoustic emissions. This section summarizes only the equipment used to measure the parameters reported in this work and detailed below. Arrangement of the monitoring system is shown in Figure 3.

The layout of the monitoring system has been defined according to the load test setup, considering two main zones: the shear span and the bending span. Along the shear span, linear variable displacement transducers (LVDTs) were installed on aluminum frames oriented at 45° with a measurement base of 707 mm. Such sensors were denoted by the code “SHxxA/B”, where “xx” indicates the progressive number, “A” refers to the frames with a negative slope, and “B” refers to the frames with positive slopes. In the bending zone, the LVDTs were installed on horizontal frames with a measurement base of 400 mm. Those were denoted by “BxxT/C” where “xx” indicates the progressive number, “T” refers to the LVDTs installed on the bottom flange, and “C” refers to the remaining positions. Furthermore, two electrical strain gauges were mounted on the bottom and upper flanges of the precast beam along the left vertical alignment of the B01T and B01C LVDTs.

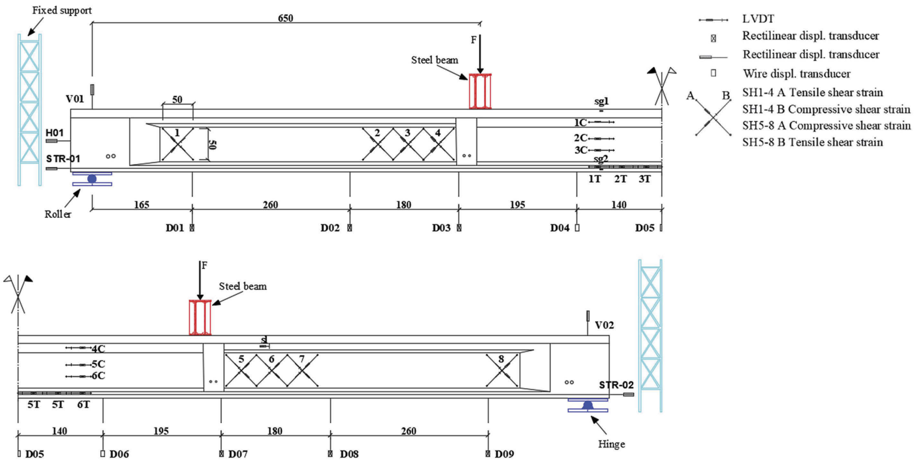


Figure 3. Layout of the monitoring system (units in cm).

Three displacement transducers were installed at each end of the girder to measure vertical and horizontal displacements at the supports, and the potential slipping of one strand. A displacement transducer was installed along the shear span to measure potential sliding of the cast-in-situ slab. The vertical deflections along the beam were measured by nine potentiometer transducers connected to the bottom of the girders (“D01-09” in Figure 3). For the B7-P47/46 girder, the LVDTs along the shear span, in positions 2-4 and 5-7, were moved 50 cm from the transverse diaphragms. The LVDTs in positions B02/03C and B05/06C were used along the shear span, close to positions 1 and 8. The two strain gauges were moved to the same alignment as the B03T. For B10-Ab/P47 girder, the two strain gauges have been moved to the midspan.

4 TEST RESULTS

Some of the most representative results of the full-scale load tests have been reported in the present paper. These diagrams were chosen to show the overall structural response of the girder as a consequence of simultaneous shear and bending actions during the mechanical tests. All the diagrams have been obtained by referring to the load applied by a single jack. Therefore, to consider the total load applied to the specimen, the load should be multiplied by 4. Furthermore, the self-weight and the weight of the loading system (estimated as 11.82 kN per actuator) need to be considered.

The Figure 4 presents the midspan deflection versus the applied load. The diagrams highlight a similar behaviour for all the specimens, which can be described by three phases: a linear elastic relationship, a cracking-induced reduction of the stiffness and the failure after reaching the maximum load due to the crushing of the upper slab.

The failure mode was similar with that registered in the first group of specimens (Savino et al. 2023). However, a different resistance mechanism was involved, as it will be shown in the following based on the crack pattern.

The B7-P47/46 reached its maximum capacity with a load of 120.1 kN, inherent to an applied bending moment of 1561.3 kNm. The B6-Ab/P47 showed a failure load of 126.5 kN; this load

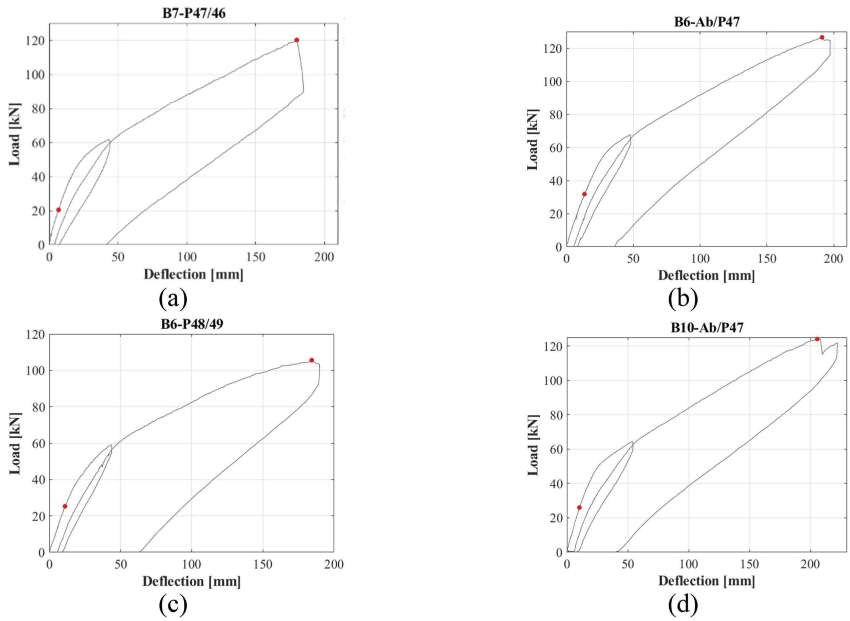


Figure 4. Load vs deflection: (a) B7-P47/46, (b) B6-Ab/P47, (c) B6-P48/49, and (d) B10-Ab/P47 girder.

corresponds to a maximum applied bending moment of 1644.5 kNm. The B6-P48/49 was loaded up to a failure load of 105.6 kN, corresponding to a bending moment of 1372.8 kNm. The B10-Ab/P47 was loaded up to a failure load of 124.0 kN, corresponding to a bending moment of 1612.0 kNm. At these load levels, the B7-P47/46, B6-Ab/P47, B6-P48/49, and B10-Ab/P47 specimen exhibited maximum midspan deflections of 179.9, 191.3, 184.3 and 205.5 mm, respectively. Referring to the load-midspan deflection diagram of the specimens tested under three-point loading configuration, the maximum bending moment and midspan deflection were 1492.2 kNm (B8-P47/46) and 167 mm (B3-P47/46) respectively (Savino et al. 2023).

Since the load-deflection curves are function of the global behavior of the specimens, the first cracking load was detected from the load-tensile strain curves recorded by local transducers installed at the bottom flange of the PC girders near the midspan. The cracking load was determined in the sections that firstly began to deviate from the linear trend. As an example, Figure 5a shows a comparison between the LVDT readings and the strain gauge sg2, which shows the lowest cracking load. The estimated cracking loads for B7-P47/46, B6-Ab/P47, B6-P48/49, and B10-Ab/P47 beam were 20.7 kN, 25.5 kN, 17.2 kN, and 22.1 kN, respectively.

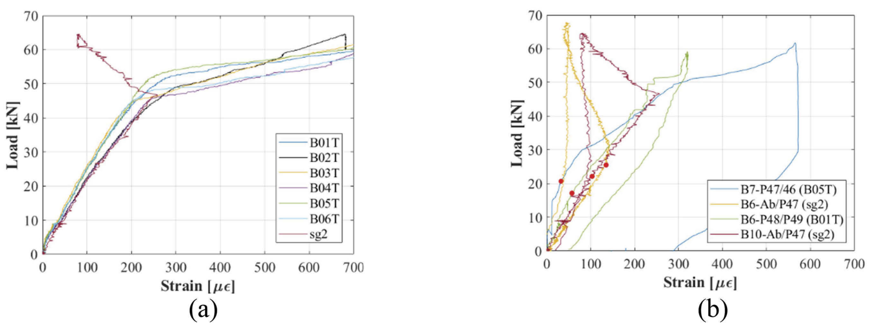


Figure 5. Load versus tensile strains: (a) signals comparison for B10-Ab/P47, (b) cracking load for B7-P47/46, B6-Ab/P47, B6-P48/49, and B10-Ab/P47 girder.

All curves start with a linear branch followed by a change in strain distribution when the applied load determines the first cracking. For B6-Ab/P47 and B10-Ab/P47 specimen, as the tensile strains were recorded by the strain gauges, a reduction in strain is observed: this can be associated to a crack opening in the nearby of the strain gauge location.

The compressive strains recorded in the top region have been reported in Figure 6. The green curves refer to the strain of the cast in situ slab recorded by strain gauges, and the other two curves refer to the top flanges of the I-beams, according to the layout in Figure 3. Furthermore, the images of the failure zones are also reported. The maximum compressive strain recorded on the top slab of the B7-P47/46 beam was -2.2‰. From Figure 6b, the failure occurred close to the strain gauge position in the midspan. For the other specimens, the compressive strains on the top slab resulted -1.4‰, -1.1‰, and -1.7‰, respectively.

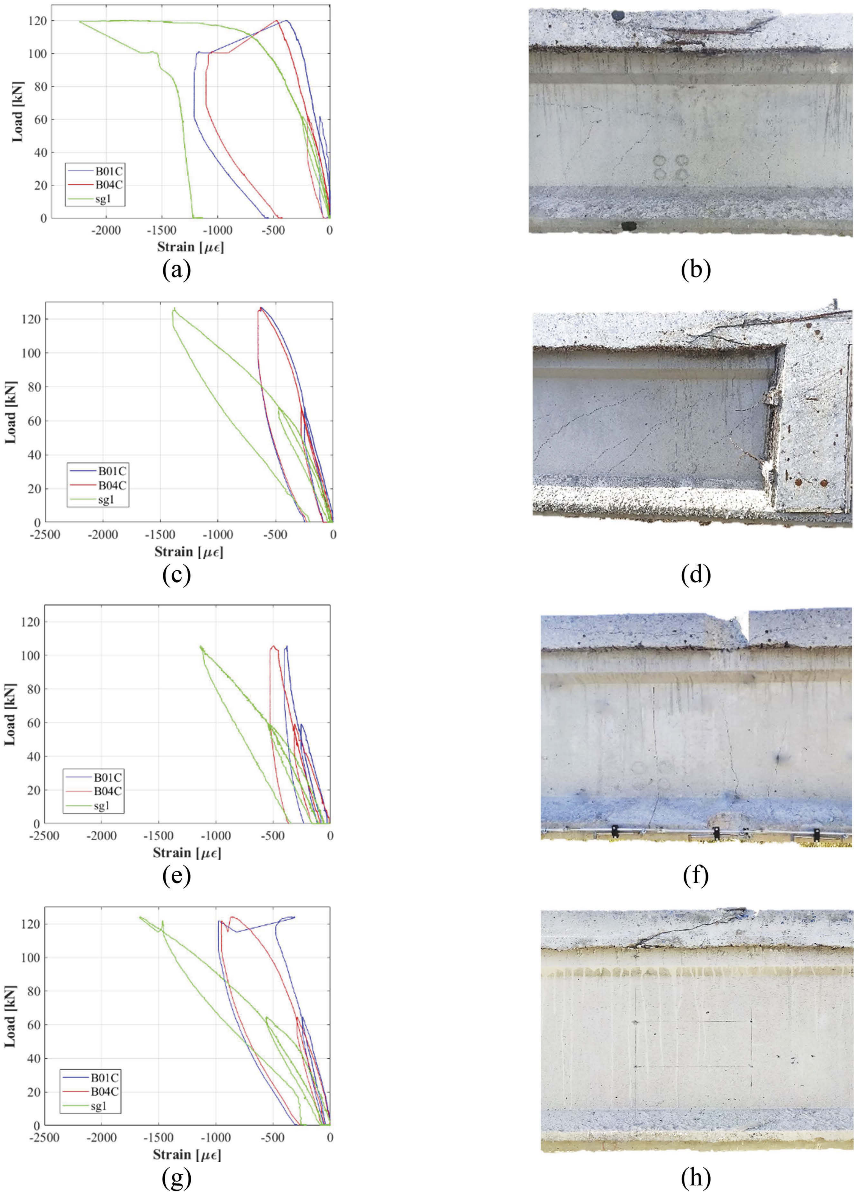


Figure 6. Load versus top compressive strain and failure zones: (a)-(b) B7-P47/46, (c)-(d) B6-Ab/P47, (e)-(f) B6-P48/49, and (g)-(h) B10-Ab/P47 girder.

During the loading process, the average strains recorded by the LVDTs installed on the diagonal frames along the shear span were considered representative of the shear strains. As an example, the shear strain curves recorded on the left side (Figure 3) of the B7-P47/46 beam are shown in Figure 7. According to these curves after the load reaches approximately 50 kN, diagonal cracks formed with an inclined angle from the longitudinal axis, and basically orthogonal to the principal tension direction.

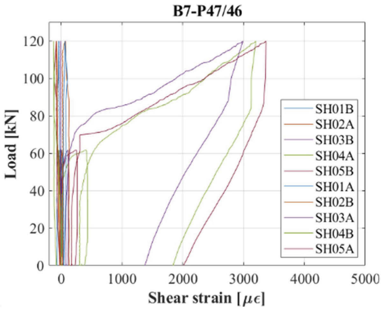
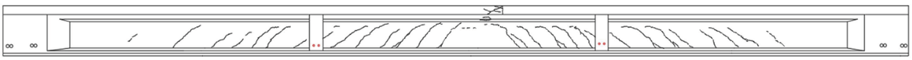


Figure 7. Load versus shear strain for B7-P47/46 beam.

The development of flexure-shear cracking for beam B7-P47/46 as representative of the four tests is reported in Figure 8a. The cracking pattern registered for girder B8-P47/46, as a representative of the previous three-point bending configurations, is reported in Figure 8b which reveals shear cracking along the whole length of the beam. Figure 8a reveals no shear strains in the constant moment region and a higher number of diagonal cracks in the shear span.

(a)



(b)

Figure 8. Crack pattern after the load tests: (a) B7-P47/46 (4-P bending test), and (b) B8-P47/46 (3-P bending test) girder.

5 CONCLUSIONS

An extensive experimental investigation has been conducted to study the structural behavior of 50-year-old PC girders at both service and ultimate load levels. The present paper summarizes the experimental part of the investigation concerning the tests involving both flexural and shear mechanisms. Specimens with different level of damage were tested, undamaged and damaged by dismantling operations, resulting in strands cut. The test results were analyzed considering deflections, strains, and cracks evolution, as well as ultimate capacities. Furthermore, the structural responses were compared with the results of specimens previously tested in three-point bending configuration. All the tests have shown a flexural compression failure with brittle crushing of the cast in situ slab.

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