Load tests on dismantled 50-year-old prestressed concrete bridge deck beams

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ABSTRACT: This paper presents the results of three large scale experimental tests within the survey conducted on 50-year-old prestressed concrete (PC) girders for the BRIDGE|50 research project (www.bridge50.org). The girders were retrieved from a viaduct before dismantling operations once located in the urban area of Turin, Italy. Together with other structural members they represent one of the most prominent project of this type worldwide. The PC elements have 19.2 m span length and I-shaped cross section with 14 cm cast-in situ slab with variable damage due to deterioration caused by both service and lifting operations during dismantling phase. For each specimen, static tests have carried out applying monotonic or cyclic loading up to the ultimate load, measuring deflections, loads and strains in several positions for two I beams and one box beam. The results of the tests in terms of load-deflection responses and strains are reported for each beam investigated and compared considering previous tests from the same project already reported in literature. The experimental findings highlight the specific structural response and residual capacity of the tested members in presence of damage, cyclic loading and different tested section. The outcomes of the project will provide a valuable database of reference for the assessment of the residual structural performance of existing bridges.

1 INTRODUCTION

Bridges in service are subjected to environmental exposure which is responsible of durability problems. The impact of these aggressive attacks may involve a significant reduction up to the total loss of load-carrying capacity. Moreover, traffic demand is continuously growing, both in terms of number and weight of vehicles. In addition, as civil infrastructures age, uncertainties on the prediction of structural response increase due to the deterioration mechanisms and their scattered nature (Ellingwood 2005, Biondini & Frangopol 2016). As a result, structural assessment has received much attention in recent decades from different perspectives, including structural health monitoring techniques employed to collect data for structural condition evaluation and early warning detection. Most of the experimental research activity in this field has been done investigating laboratory specimens, under controlled environment. However, full-scale load tests on existing structures are fundamentals to better understand the structural behaviour and to calibrate structural models. The latter applies to models from structure to material scale that are even more required for safety evaluations (Franceschini et al. 2022, 2023). But, large-scale tests on civil infrastructures, are not widely performed also because they involve irreversible damages which led to the dismissing of the structure or part of it. 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.

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However, the investigations usually concern small samples of structural members and basically focusing on few characteristic parameters of the structural response. A systematic assessment of a number of specimens under different loading test setups is required to investigate the failure mechanisms and thus developing realistic estimates of the residual capacity. Moreover, this survey allows to take into account the influence of coupled precast and cast in situ concrete parts which work together in the final configuration.

BRIDGE|50 research project was specifically defined to study the residual structural performance of 50-year-old viaduct members 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 PC I-girders, PC box girders, and pier caps recovered from the C.so Grosseto viaduct, a viaduct built in Turin in 1970 and decommissioned in 2019. Updated information on the BRIDGE|50 research program is posted on the project website (www.bridge50.org). Multiple load test setups were considered to study flexural, flexural/shear, and shear failure mechanisms. 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. (2023a). The second set of members, subjected to a four-point bending test setup are reported in Savino et al (2023b). The present paper reports the experimental results of the third group of three 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 setup and calibrate mechanical models.

2 CORSO GROSSETO VIADUCT

The Corso Grosseto viaduct was a road bridge located in Turin, Italy (Savino et al. 2020). The structural scheme of the PC viaduct was with simply supported spans: a widespread solution in Italy and around the world. The deck was composed by ten PC I-shaped girders and two PC box girders at the edges, with a support distance between 16.0 and 24.0 m and two diaphragms connected the longitudinal girders at the third points (see Figure 1). The superstructure was completed by a cast-in-situ slab of 140 mm.

An overview of the retrieved structural members in their original deck configuration is shown in Figure 1. The elements coloured represent the collected ones; the girders depicted in blue belong to the first and second group of tests whereas the elements highlighted in red refer to the girders investigated and reported in the present 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 B5-P46/47, BX2-Ab/P47, B2-P46/47 according to classification reported in Savino et al. (2023).

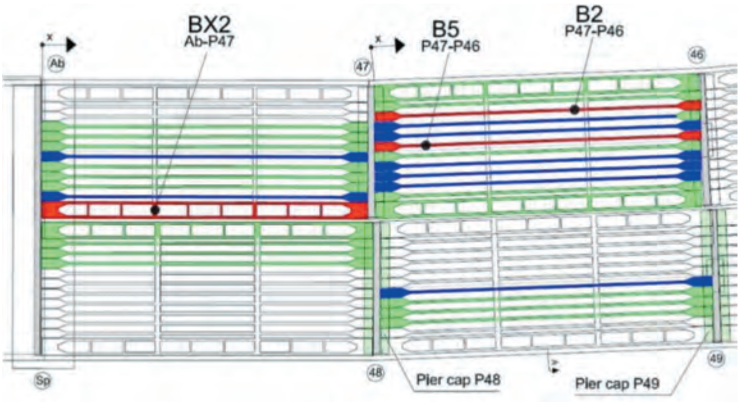

Figure 1. Layout of the retrieved and studied elements (green: beams to be tested; blue: previously tested and reported in other papers; red: investigated in this paper).

2.1 *Tested girders details*

All the girder beams were 19.5 m long, with I-shaped cross-section as shown in Figure 2a for beams B5-P46/47 and B2-P46/47 and box-section as shown in Figure 2b for beam BX2-Ab /P47 (from design documentation). Cover depths were 45 mm for prestressed strands and 10 mm for ordinary reinforcement. Prestressing reinforcement was composed of seven-wire strands with a nominal diameter of 12.7 mm and according to the design documentation, the tensile strength was of 1638 MPa. The prestressed reinforcement configuration consisted of 17 strands for I-beams and 34 strands for box-beam, located in the bottom flange. Additionally, there were 3 strands for I-beams and 6 strands for box-beams located in the top flange. 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 pattern 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.

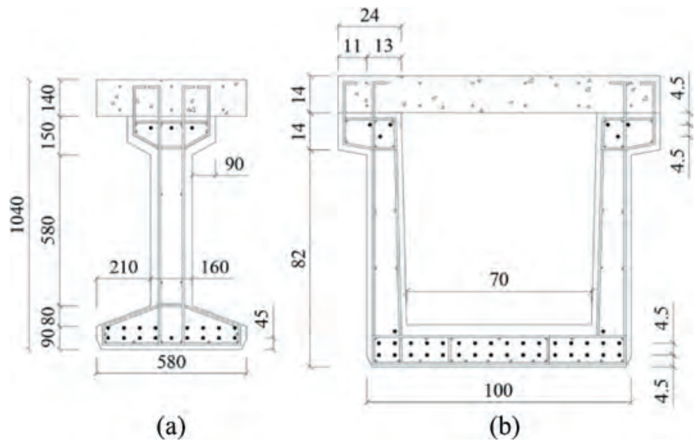

Figure 2. Cross-section of the beams (units in mm): (a) I-beam; (b) Box-beam (from design documentation).

According to the diagnostic campaign performed before the load tests session, the beams investigated in this paper were considered in good conditions. Beam B5-P46/47 was intentionally damaged by cutting 8 strands at the midspan to induce a different failure mode from that observed in previous tests (Savino at al. 2023b), as depicted in Figure 3. The damage was created by cutting a total of eight strands in the bottom flange at a distance of 60.0 cm from the girder midspan on either side.

Girder BX2-Ab/P47 had the curb on the top slab; in order to avoid torsional effect, a preventive regularization of the upper surface was carried out by cutting it with a diamond blade.

3 SETUP OF THE FULL-SCALE LOAD TESTS

For the purpose of testing the PC girders, a proper reaction steel frame was made available by the Interdepartmental Center SISCON (Safety of Infrastructures and Constructions) of Politecnico di Torino. Each beam was tested under a simply supported static scheme under a four-point loading configuration adopting shear spans of 650 cm. The loading system details and the specifics of the employed equipment can be found in Savino et al. (2023a). The girders were tested according to multiple loading phases. The multiple loading cycles also allowed to perform the dynamic characterization of the specimens at different level of damages (Quattrone et al. 2020, Sabia et al. 2021, 2022, 2023). In particular, the first loading phase was

Figure 3. Strands cutting in B5-P46/47 girder (bottom view).

meant to reach the stabilized cracking pattern to evaluate the residual prestressing during the following crack reopening phase.

In detail, B5-P46/P47 was tested with two loading cycles, Bx2-Ab-P47 was tested with three cycles of which the second cycle had maximum loading corresponding to half of the expected capacity; finally, B2-P46/P47 was tested with three sets of five cycles each before reaching the final stage up to failure. This choice was motivated by the intention to induce the effect of cumulative damage at different levels and collect the effects on both the dynamic behaviour and residual resistance of the girder. All load tests were performed with a fixed loading rate under force control and were stopped upon the occurrence of failure, as indicated by load reduction detected by the load cells and confirmed by evident damage recorded by the measuring devices. The setup of measuring devices for girders B5-P46/P47 and B2-P46/P47 are reported in Savino et al. (2023b). In these tests the equipment was placed on one side of the beam; in particular the side protected by the insulation was chosen and it also corresponded to the one under direct vision from the control station. Instead, the setup for the box girder BX2-Ab/P47 is reported in Figure 4, where the pattern of devices installed on the web is mirrored on the opposite side (not reported in Figure 4). This layout was selected to monitor both webs of the box girder and identify any asymmetrical behaviour during the test.

For all the tests, the arrangement of devices has been defined according to the load test setup, considering two main zones: the shear span and the bending span. Along the shear span, specifically near the transverse beam, linear variable displacement transducers (LVDTs) were installed on aluminium frames oriented at 45° angle, with a measurement base of 707 mm. Such sensors were labelled with 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, specifically across the midspan, the LVDTs were installed on horizontal frames with a measurement base of 400 mm. They 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, four electrical strain gauges were mounted as follows: on the top and bottom flanges of the precast beam along the left vertical alignment of the midspan, on the cast-in-situ slab, and the last on the prestressing strand, at 120 cm away from the midspan. 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 a strand. Another displacement transducer was place along the shear span to measure potential sliding of the cast-in-situ slab. The vertical deflections along the beam were measured using nine potentiometer transducers connected to the bottom of the girders (“D01-09” in Figure 4).

4 TEST RESULTS AND DISCUSSION

This paper includes a subset of the comprehensive results obtained from the full-scale load tests. These diagrams were chosen to show the overall structural response of the girders

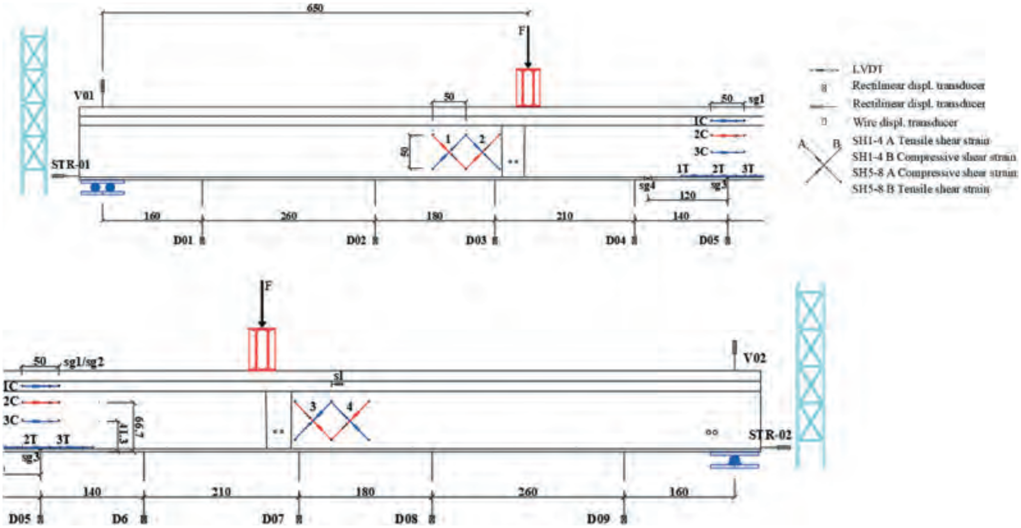

Figure 4. Layout of the testing system with measuring devices for girder BX2-Ab/P47 (units in cm).

together with some specific readings during the tests. All the diagrams have been obtained by referring to the load applied by a single jack. Therefore, the total load applied to the specimen can be found multiplying by 4. Figures 5a-c present the midspan deflection versus the applied load for the three girders. In Figure 5d a focus of the first two sets of cycles for girder B2-P46 /P47 is reported.

A global overview confirms for the specimens a structural behaviour defined by three phases: a linear elastic branch, a concrete tensile cracking that induced stiffness reduction and brittle failure after reaching the maximum load. The girder B5-P46/P7 in Figure 5a shows earlier non-linear behaviour attributable to the cutting of 8 strands across the midspan and this obviously anticipate the first cracking and determines the reduced initial stiffness of the beam. This is evident if comparison is made with previous test results reported in Savino et al. (2023b). All the three specimens reached the collapse due to crushing of the cast in situ top slab. Beam B2-P46 /P47 completely failed and followed down to the safety support system of the reaction frame whereas the test for the other two girders was stopped before global collapse.

The failure mode was similar with that registered for the previous girders tested under four-point bending configuration (Savino et al. 2023a 2023b). As previously stated, beam B5-P46/47 was intentionally damaged to induce a more ductile behaviour than what observed for other identical beams reported in Savino et al. (2023b). The ultimate loads were 72.4 kN, 241.5 kN and 126.2 kN for beams B5-P46/47, BX2-P46/P47 and B2-P46/47 respectively. The structural capacity reduction for beam B5-P46/P47 with respect to an identical girder named B6-Ab/47 and reported in Savino et al. (2023b) but with undamaged strands corresponds to a 42.8%.

The BX2-P46/P47 shows a higher stiffness due to its box section made with two webs of 130 mm and the double number of strands used in comparison with I girder B2-P46/47 reported in this work (see Figure 2). A residual deflection registered after the cycles evidenced the role of non-linearities also due to tensile cracking and possible mechanical interlock. The test on girder B2-P46/P47 was specifically meant to produce progressive damage by cycles of increasing load levels. It can be noted from Figures 5c and 5d that most of the damage was determined by the first cycle of each set whereas small additional damage has been registered for the other four cycles. A progressive residual deformation is visible but, even in case of the last spare cycle before the last branch bringing to collapse, its descending branch was almost aligned with the previous readings without evident stiffness reduction.

Figure 6a shows the strain distribution measured across the midspan along the cross section (see horizontal transducers type in Figure 4) at various loads for girder B5-P46/P47. According to the diagram of the first two steps (10 kN and 20 kN) the section has a plain

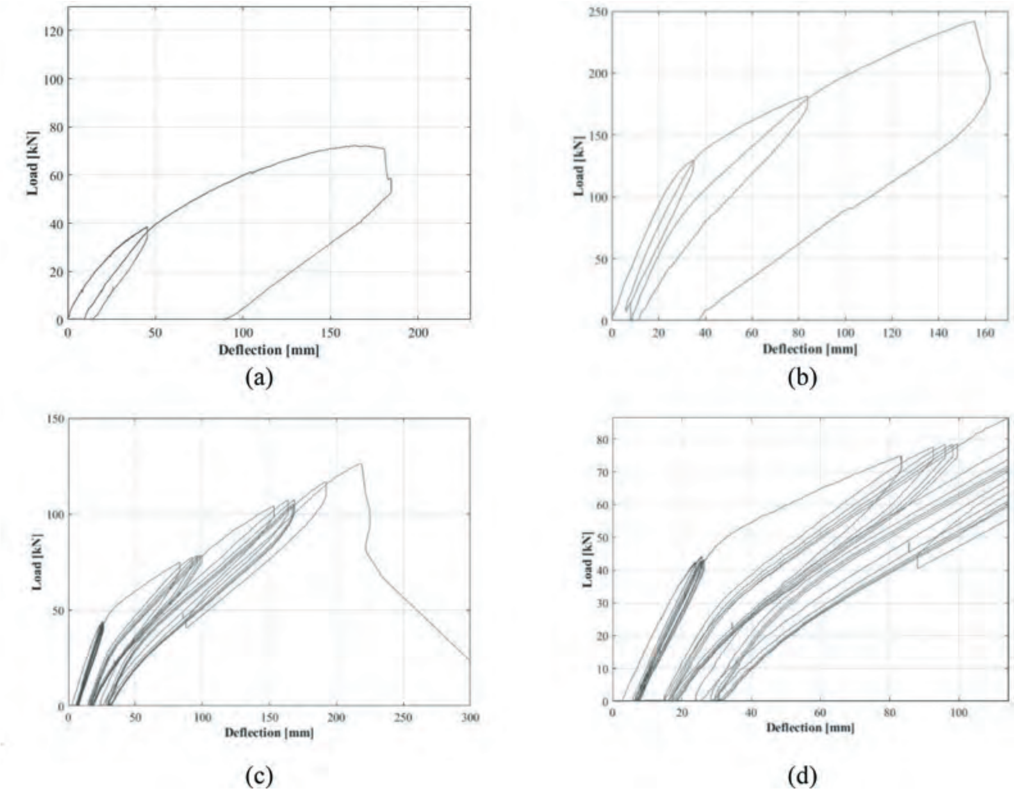

Figure 5. Load vs deflection diagrams: (a) B5-P46/47; (b) BX2-Ab/P47; B2-P46/47: (c) full results and (d) detail.

deformation. At cracking stage the effective depth of the compressive zone was about 0.49 m. When cracking occurred, the neutral axis shifted up and the strain distribution became non-linear. At ultimate condition the neutral axis reached the value of 0.23 m and therefore only the cast-in-situ top slab with a part of the top flange of the precast beam were in compression. The same diagram is reported in Figure 6b for girder B6-Ab/P47 (undamaged in Savino et al. 2023b) that had, even up to collapse, a deeper neutral axis. This highlights the effect of the full prestressing pattern for undamaged girder. Finally, the type of collapse for girder B5-P46/P47 that had 8 strand cut highlight the limited ductility even in presence of around 47% of the tensed steel in the section.

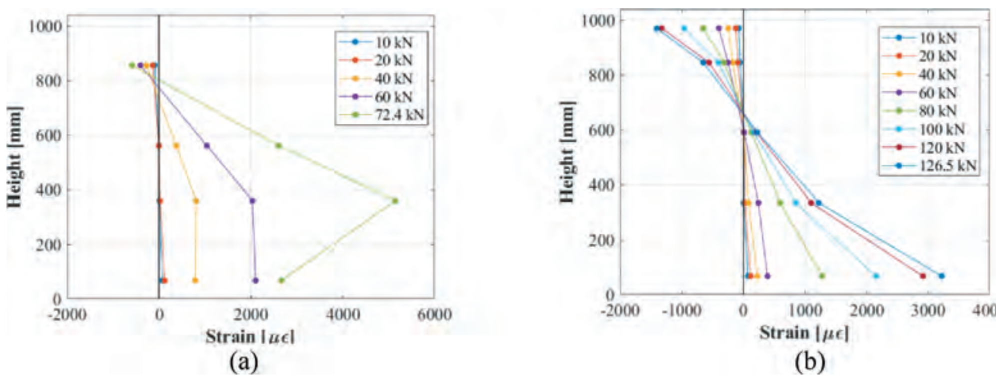

Figure 6. Cross-section strain sg1: C-01, C-02, C-03, and T-01 of B5-P46/47 and (b) B6-Ab-P47.

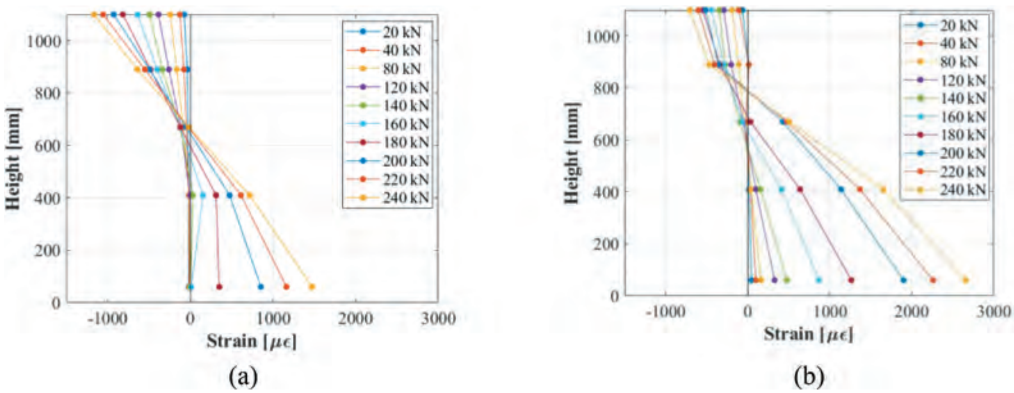

Figure 7. Cross-section strain of BX2-Ab/P47 girder: (a) inner side; (b) exposed side.

Due to the different exposure and consequent damage, the box girder BX2-Ab/P47 was under investigation for what concerns the strains distribution on both the two webs. In Figure 7, the strain distributions are reported. A noticeable difference between the inner side (Figure 7a) and the exposed side (Figure 7b) is observed, becoming more evident beyond a load level of 160 kN. At this stage, a larger shift of the neutral axis is recorded by the outer web. At ultimate condition, the ratio between the depth of the compressive zone and the effective depth were 0.63 and 0.52 for the inner and exposed web, respectively. Therefore, a non-symmetric behaviour can be noted with major deformation attributable to the web with much visible damage (see Figures 8a,b).

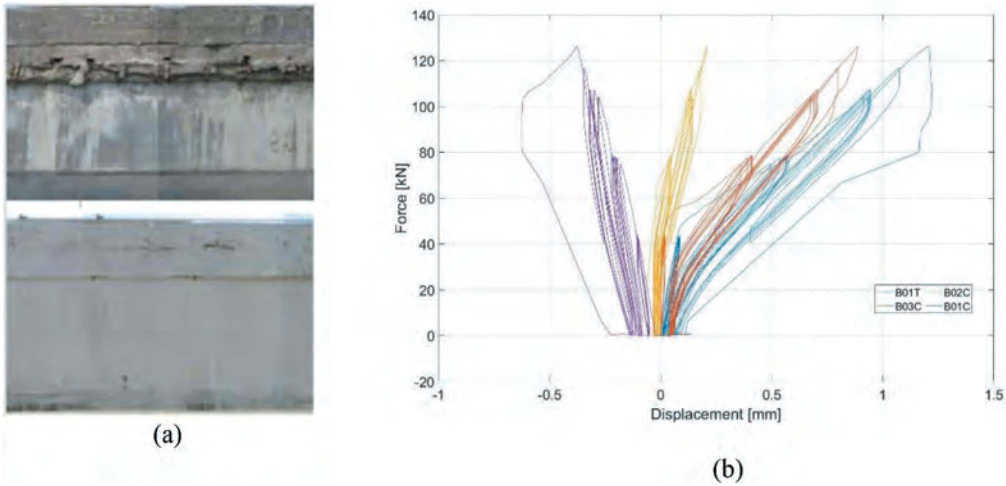

Figure 8. (a) Comparison of the inner (bottom) and exposed side (top) of the beam Bx2-Ab/P47; b) Load versus displacements of the transducers used for tension (T) and compression (C) for girder B2-P46/P47.

In Figure 8b the load-displacement diagrams of four transducers in the top flange (B01C), web (B02C, B03C) and bottom slab (B01T) of the beam B2-P46/P47 are reported. These devices were installed across the midspan and therefore in the most loaded zone of the girder. It is evident that after the first set of cycle for curve of B01T, a cracking occurred and a permanent modification with slope reduction of the curve is registered; all the other three devices, measured the displacements and report slope of the curve similar to the first loading up to the last cycle. B01C and B01T as devices placed in the most compressed and tensed zones respectively show some residual displacement. It can be stated that, even for the most loaded zone of the beam the accumulated damage resulted to be very limited up to brittle failure in crushing of the cast-in-situ top slab.

5 CONCLUSIONS

An experimental activity on bridge girders has been conducted to study the structural behaviour of 50-year-old PC members at both service and ultimate load levels. Three large-scale experimental tests have been reported. Specimens with different level of damage, geometry, and loading protocol were tested. The results have been analysed considering deflections, strains as well as ultimate bearing capacities. All the tests reported in this work have shown a flexural failure with brittle crushing of the cast-in-situ slab. These findings can be further analysed and compared with the outcomes obtained for the previous bridge girders tested under the BRIDGE|50 research project highlighting a wide number of peculiarities of the residual capacity of PC bridge girders.

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