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Full-scale testing and analysis of 50-year old prestressed concrete bridge girders

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ABSTRACT: The present research covers the full-scale loading tests conducted on 50-year old prestressed concrete girders. The beams were removed from an existing viaduct in Turin, Italy, and gave the possibility to setup a large experimental campaign as part of the research project named BRIDGE|50 (www.bridge50.org). The precast prestressed concrete beams have approximately 19.2 m span length and I-shaped cross section with 14 cm cast-in situ slab. For each specimen, static tests have carried out applying monotonic and cyclic loading profiles up to the ultimate load, measuring deflections, loads and strains in several positions. The results of the tests in terms of load-deflection responses and strains are reported for each beam investigated and compared each other. The experimental findings highlight the global structural response and the outcomes will define a reference for future studies aimed to assess the residual structural performance of existing bridges.

1 INTRODUCTION

Bridges are key structures in the transportation infrastructures networks of a country that ensure productivity, growth and quality of life. During their designed service life, usually intended as 50 years, these structures are subjected to many aggressive influences, such as variable loadings and vibrations, extreme weather conditions, presence of chlorides, freeze and thaw cycles, which can impair their structural behavior. Most of the existing bridges were constructed from the 1950s onward, when the development of the precast and prestressed concrete industry allowed to satisfy the needs of speed in construction, design flexibility, enhanced performance and maximum economy. As a result, the oldest of these passed the end of their design life and most of them need to be refurbished or even to be replaced. The management of old infrastructural assets have become the 21st century challenge for transportation agencies around the world, which are engaged to maintain healthy infrastructure and prevent unexpected structural failures considering cost-effective planning. Bridge managers need to prioritize limited budgets to cost-effectively maintain the normal functionality of a huge inventory of deteriorating civil infrastructure such as highway bridges over their entire life-cycle (Biondini & Frangopol 2016, 2019).

According to the Report Card published by the American Society of Civil Engineers in 2021, 42% of all US bridges are at least 50 years old and 7.5% are considered structurally deficient (*ASCE 2021*). Recent estimates report that, at the current rate of investment of $14.4 billion per year, it will take about fifty years to finalize all the repair interventions that are currently necessary. Similar situations can be found in other countries (*ICE 2014*, Australia's Local Government 2021). The European Construction Industry Federation (FIEC) warned that 60% of the post-war EU structures warns about corrosion of steel and these issues are even worsen by a lack of funds for maintenance (European Commission 2019). According to recent estimations of the National Association of Corrosion Engineers, the global cost annually spent on corrosion maintenance and rehabilitation is about the 3.4% of the Gross Domestic Product of industrialized countries, that is $2.5 trillion (*NACE International 2019*).

These statistics highlight the urgent need to assess safety and functionality of bridges. The high financial investments required by retrofitting works make the extension of the service life the only viable choice in the short-term. Immediate action is needed to recognize and assess current risks, predict performance and implement rational management strategies to maintain structural performance within acceptable levels throughout the life-cycle of deteriorated bridges.

Currently, deterioration levels and the residual safety related to service life are typically estimated on the basis of periodic visual inspections, assigning the corresponding Bridge Condition Indexes in Bridge Management Systems. However, there are potential drawbacks in the visual inspection process related to subjectivity and uncertainty related to inspection procedures. Furthermore, there are hidden defects such as internal cracks, rebar corrosion, voids and long-term effects (i.e. creep, shrinkage and relaxation) which are not simply detectable by inspectors but which can deeply affect the structural performance. In some cases non-destructive evaluation techniques may be also used allowing to overcome such limitations. However, for a proper structural evaluation of aged structures it is required an in-depth knowledge concerning the long-term performance of existing bridges continuously exposed over their lifetimes to aging of materials, fatigue and degradation phenomena.

In the present study, the outcomes of loading tests on three prestressed concrete (PC) beams recovered from a 50-year-old bridge deck are reported. These tests are part of a wider experimental campaign of the BRIDGE|50 research project, aimed at investigating the residual structural performance of a decommissioned 50-year-old road bridge. The aim is to collect new data from inspections and experimental tests to develop new methods for safety assessment, most cost-effective maintenance, and management of existing infrastructure (Biondini et al. 2021a, 2021b, 2022). The beams were removed from the C.so Grosseto Viaduct in Turin, Italy. The construction of the bridge was completed in 1970 and in September 2018, its deconstruction was started due to the new urban redevelopment and different mobility needs (Savino et al. 2021). During the demolition works, a group of 29 PC deck beams and two pier caps were retrieved by a saw-cutting of the slab and transverse beams and stored in a testing site. For more details on the dismantling and demolition operations and the setup of the testing laboratory refer to Anghileri et al. (2021a). The study discussed here concerns the evaluation of in-service load behavior and ultimate load for three I-shaped PC beams.

The following sections describe respectively the characteristics of the bridge deck beams, the load tests performed and the experimental results obtained. Finally, the observed results are discussed.

2 DESCRIPTION OF THE DECK BEAMS

The beams considered herein were recovered during the demolition of the 80-span C.so Grosseto Viaduct composed of precast PC girders with cast-in place deck. Each girder was composed of ten inner precast PC I-beams and two external precast PC U-beams, simply supported at piers and connected by two transverse beams. In Figure 1, the beams retrieved for investigations under the BRIDGE|50 research project are highlighted in the original position on the decks. The beams analyzed in the present study, respectively named B3-P47/46, B4-P47/46 and B8-P47/46, are depicted in red. Each beam was approximately 19.50 m long and had a 19.15 m design span in the deck configuration.

The dimensions of the I-beams cross section completed by the in situ casted concrete top slab are shown in Figure 2 and reported in Table 1.

The beams were pretensioned with twenty 12.7 mm diameter strands characterized by an average yielding and ultimate strength values of 1486 and 1722 MPa, as reported on mechanical test certificates released in 1970s by the Laboratory for Testing Materials of Polytechnic of Turin. The design reports indicated an initial prestress force of 1400 MPa and an estimated prestress loss of 565 MPa, including elastic shortening, creep, shrinkage, and relaxation. The recorded 28-day compressive strength of the steam cured pre-

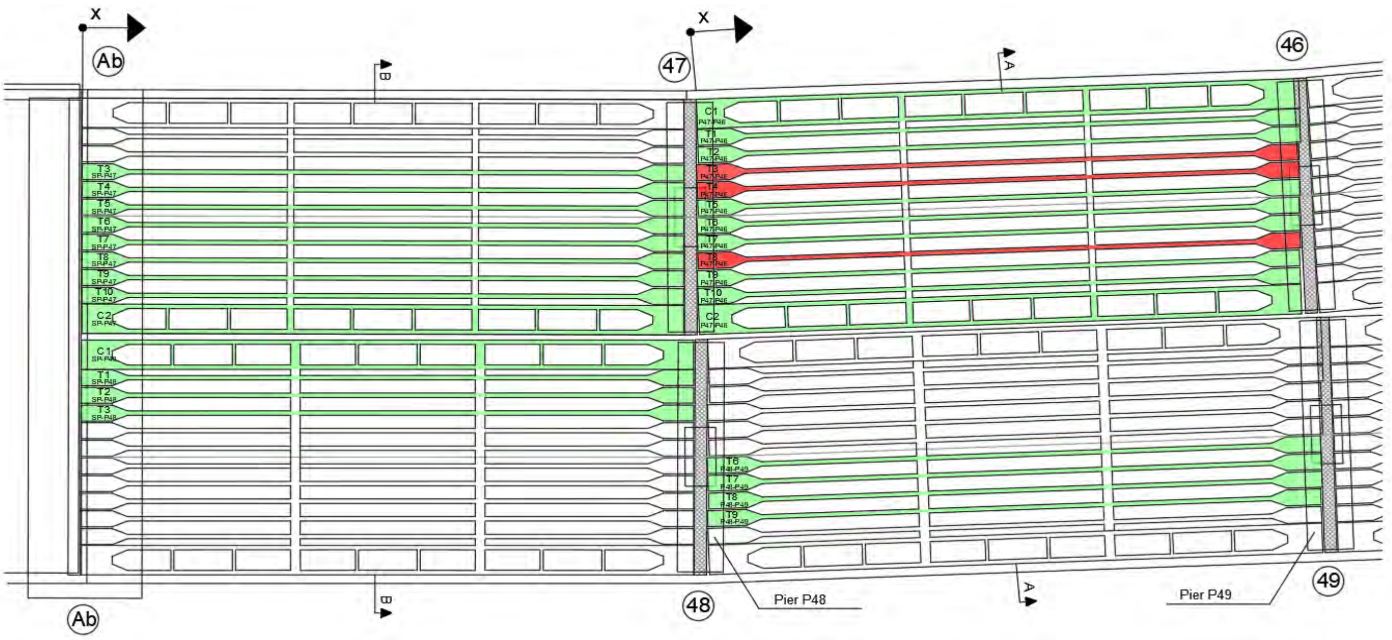

Figure 1. Original configuration.

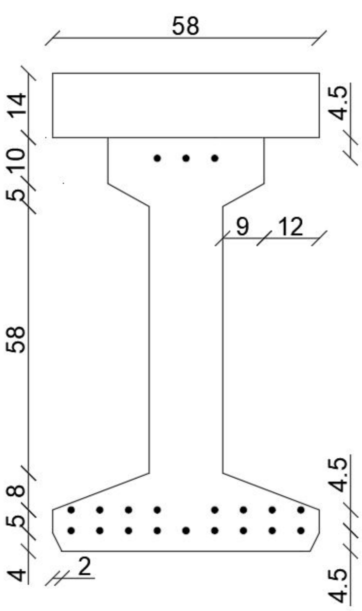

Figure 2. I-beam cross section (cm).

casted concrete was 50 MPa, whereas the resistance of the topping slab concrete was 40 MPa. In order to integrate the information provided by the available technical documentation, a wide diagnostic campaign including dynamic and non-destructive field tests have been performed (Sabia et al. 2021, Anghileri et al. 2021b, Carsana et al. 2021). According to the preliminary visual inspections aimed to map the deteriorations in terms of cracks and delamination, the three beams mentioned above were classified in good structural condition, i.e. no visible deterioration phenomena were noted.

Table 1. Section properties of I-beam.

Property	Precast beam	Composite beam
I [cm^4]	2155221	4253992
A [cm^2]	2295	3135
y_{bottom} [cm]	38.77	54.37
y_{top} [cm]	51.23	35.63

I = moment of inertia; A = cross-sectional area; y_{bottom} = distance from centroid of section to bottom of beam; y_{top} = distance from centroid of section to top of beam

3 LOADING TESTS OF DECK BEAMS

3.1 *Test setup*

The beams composed by the precast element and the overlying slab portion were stored on supports to closely replicate the original in-service configuration in a testing site in Turin. The tests were conducted by using a steel reaction frame specifically fabricated and provided by the Interdepartmental Center SISCON (Safety of Infrastructure and Construction) of Polytechnic of Turin. The reaction steel frame allows to perform 4 point bending tests allowing variable distances between the forces applied by two couples of servo-hydraulic cylinders. The Figure 3 shows a general view of the test setup. In the tests described in this work, the two loading points were applied at 0.6 m apart to generate bending failure. Future tests will examine different types of loading schemes, in order to observe the effects of shear or the shear-bending interaction. In the present study, the load refers to the action applied by 1 actuator and therefore the total applied force should be multiplied by 4.

3.2 *Instrumentation*

The structural behavior of the specimens during the loading tests is analyzed by recording several physical parameters such as displacements, strains, deflections and crack widths. Wire potentiometers were used to measure the deflections in nine points along the length of the beam. Concrete strain measurements were recorded continuously throughout the test by placing LVDTs with a base of 400 mm, on one side of the specimen, on the top and bottom flange along the central 2.8 m of the beam (Fig. 4). In particular, the latter were installed in line sharing a measurement base to detect crack opening without gaps. In addition, LVDTs were installed with an angle of 45° to collect shear strains on a base of 750 mm. The vertical and horizontal displacements at the supports were

Figure 3. General test setup.

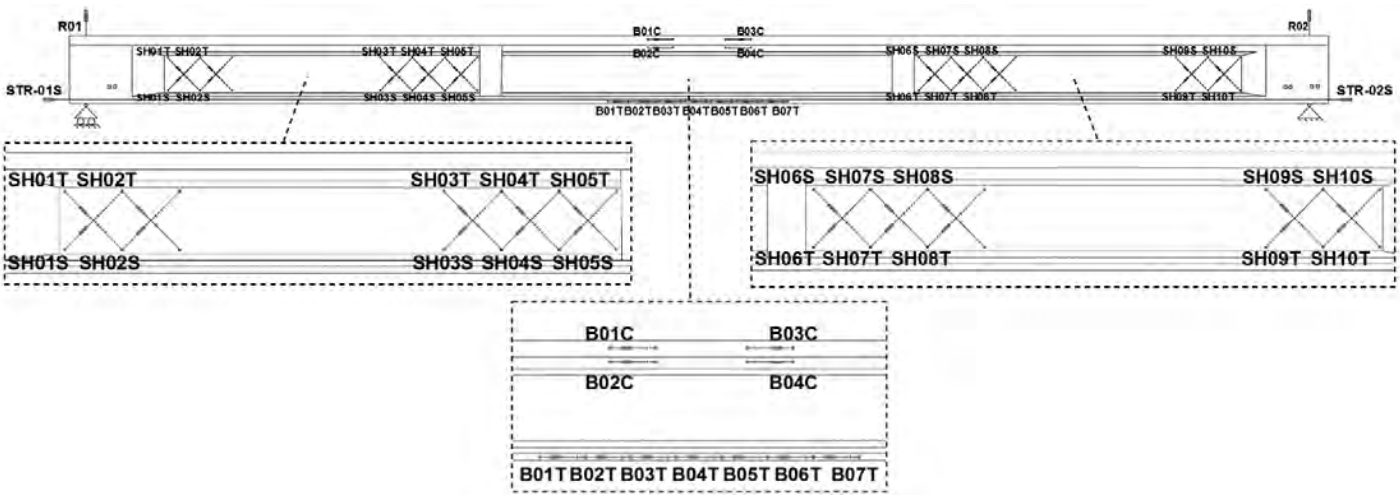

Figure 4. I-beam and instrumentation.

recorded by devoted LVDTs. Figure 4 shows the detailed scheme of the instrumentation installed on the B4-P47/46 beam. Analogous configuration was adopted for the other beams. For more details refer to Tondolo et al. (2021). The applied load was measured instrumenting one of the actuators with a load cell. The signals have been simultaneously acquired adopting a sampling frequency of 5 Hz.

3.3 *Testing procedure*

Each specimen was loaded with an average loading rate of 1.6 kN/min up to the service and ultimate phases. The B3-P47/46 and B8-P47/46 specimens were unloaded and reloaded after a first cracking cycle to estimate the loss of stiffness and the load corresponding to the cracks reopening. During the first loading cycle the B3-P47/46 beam was loaded up to 60 kN and then unloaded; the cracking cycle for the B8-P47/46 beam was performed up to 54 kN. For the B4-P47/46 I-beam, the loading test was performed monotonically up to the collapse. All the load tests were performed up to the ultimate load.

4 TEST RESULTS

The tests showed that each specimen failed by crushing in compression of the portion close to the midspan in a nonductile flexural mode as reported in Figure 5. Visual observations during the tests highlighted no separation between the precast PC beam and the cast-in place slab, ensured by the shear connectors installed in the precast concrete and integrated with the concrete slab. Diagonal cracks have been detected in the shear span, indicating the interaction between bending and shear. Around the midspan, close to the loading system, the cracks became more vertical and spaced about 26 cm. Figure 6 shows the crack pattern recorded for the B3-P47/46 beam at 75 kN, corresponding to 96% of the failure load, 78 kN, equivalent to a bending moment of 1451 kN·m. The B8-P47/46 showed a failure load of 82 kN corresponding to a bending moment of 1525 kN·m. The B4-P46/47 was

Figure 5. Failure zone: a) B3-P47/46, b) B8-P47/46, c) B4-P47/46.

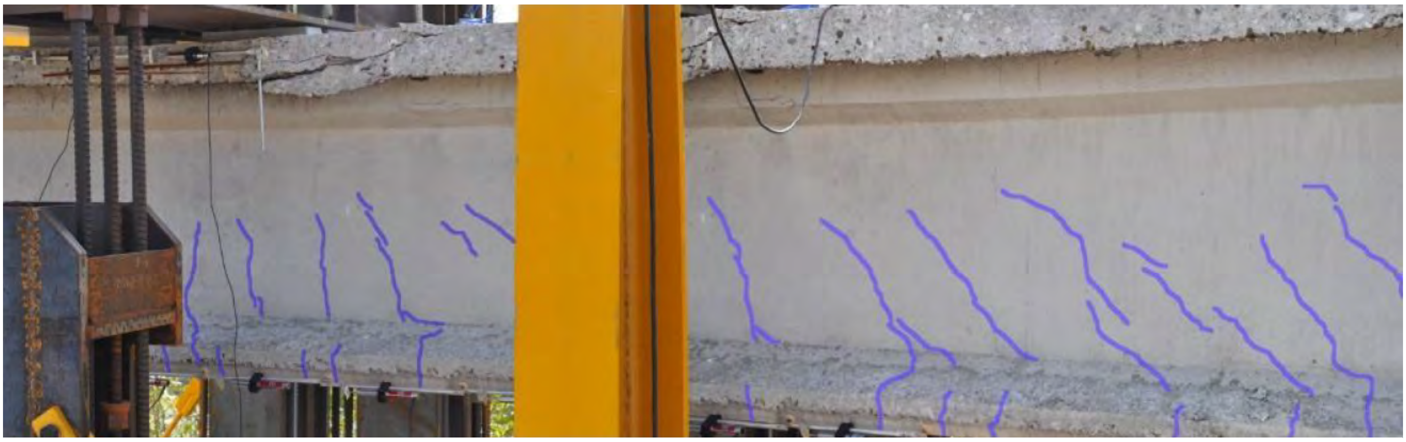

Figure 6. Crack pattern for the B3-P47/46 beam.

loaded up to a failure load of about 72 kN, corresponding to an average ultimate failure moment of 1339 kN·m. At the maximum applied load, the B3-P47/46, B8-P47/46 and B4-P47/46 specimen exhibited ultimate midspan deflections of 177 mm, 151 mm and 135 mm, respectively. The Figure 7 show the load-deflection responses of the B3-P47/46, B8-P47/46 and B4-P47/46 beams. The diagram of the first loading cycle on the B3-P47/46 beam was considered reliable up to 30 kN due to technical issues that arose during the test. The curve related to the second cycle loading shows a decrease of load at 75 kN associated to an holding stage in the test for cracking pattern evaluation. The load drop recorded for the B4-P47/46 beam at 72 kN is due to localized crushing close the loading surface.

The evaluation of the first cracks opening load cannot be clearly detectable from the load deflection diagram. Indeed, the first cracking load was detected considering the tensile strains recorded by the LVDTs installed along the bottom flange of the beams. The measured cracking load was 24 kN, 25 kN and 12 kN for B3-P47/46, B8-P47/46 and B4-P47/46 beam, respectively. All specimens showed linear elastic behavior before flexural cracking followed by a non-linear behavior highlighting a stiffness reduction. Some of the mean measured tensile strains are reported in Figure 8, which contains three LVDT signals for which the crack opening was more evident. The 2nd loading cycle, performed after the cracking for B3-P47/46 and B8-P47/46 beams, highlighted the beneficial effect of the prestressing under service loads thanks to a huge compressive strain given by the prestressing force. This leads to restore the elastic response under service conditions. Figure 9 shows the plot of LVDT measurements in the top compressive fibers. The B3-P47/46 curve shows the deformation at the top flange of the pre-casted element, whereas the B8-P47/46 refers to the deformation of the concrete slab. These curves clearly show the crushing of the top fibers of the specimens having a concrete strain of about 0.36% in the top flange of B3-P47/46 beam and 0.37% in the cast-in place slab for B8-P47/46 beam.

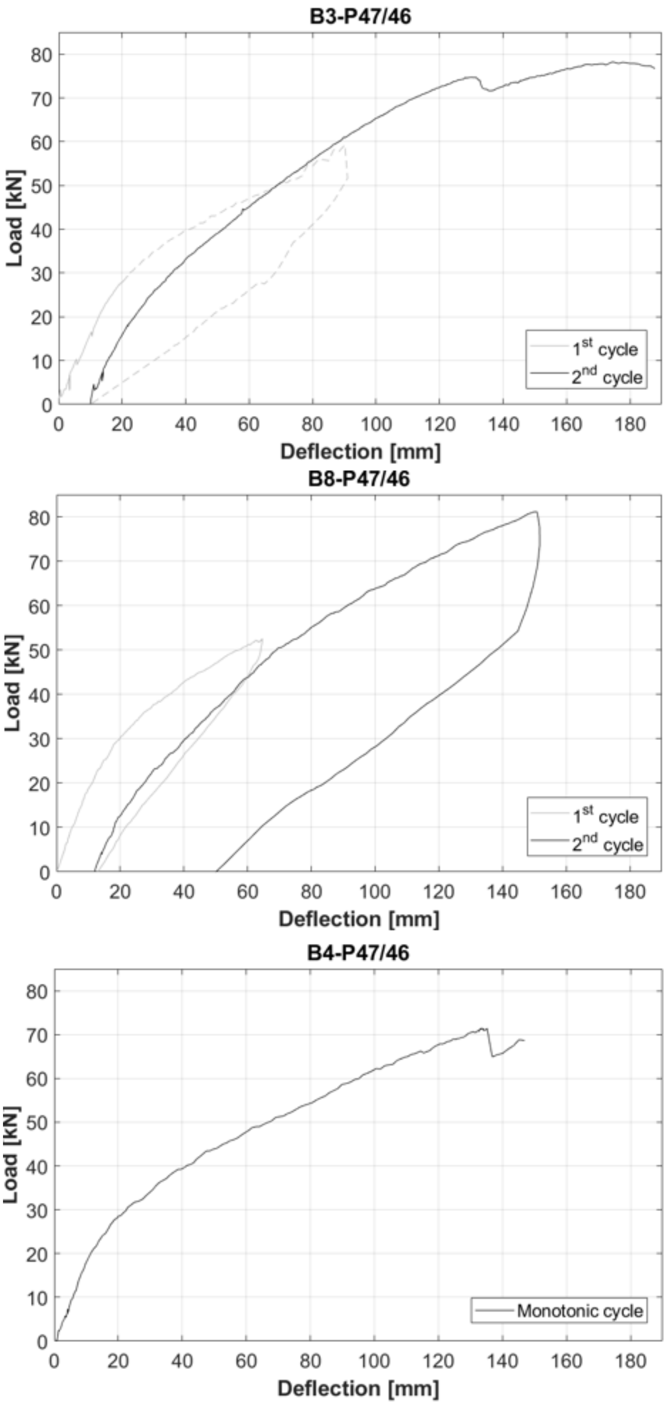

Figure 7. Measured load deflection diagram.

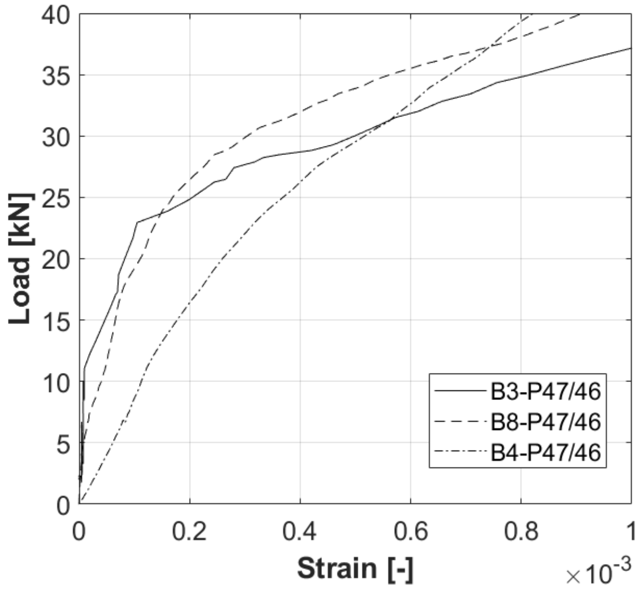

Figure 8. Load versus tensile strain.

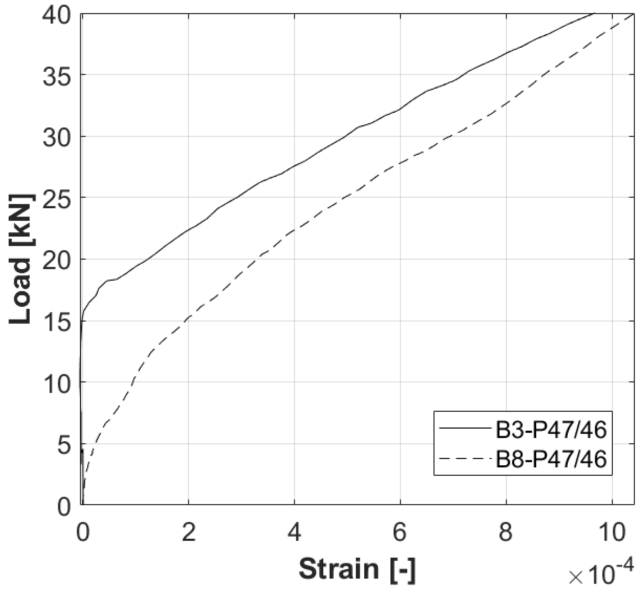

Figure 10. Load versus strain reopening.

The visual inspections performed after the loading tests showed the absence of corrosion of the strands exposed after the crushing in the failure zone. Therefore, corrosion is preliminary considered not influencing the ultimate capacity registered in the test.

In order to develop assessment techniques and predictive algorithm for life-cycle methodologies, material tests will be performed to characterize the actual mechanical properties. Furthermore, specific tests will be addressed to estimate the residual prestressing force.

A rough methodology to estimate the residual prestressing force can be obtained by analyzing the data of the second loading cycle. In particular, it can be obtained from the crack reopening load assuming that, in this condition, the stress at the location of the crack at bottom fiber is null. Therefore, the compression generated by the prestressing force at the bottom fiber is equilibrated by the loads applied on the beam and the weights of the beam and of the bench components. Figure 10 reports the LVDT readings for the reloading cycles of the B3-P47/46 and B8-P47/46 beams. Both the curves start with a first pseudo vertical branch before the re-opening of the cracks. The measurements highlight a considerable change in slope of load versus strain curves at 16 kN and 11 kN for B3-P47/46 and B8-P47/46 beams, respectively. For the B4-P47/46 beam, the reopening load has not been defined because it has been loaded monotonically up to failure.

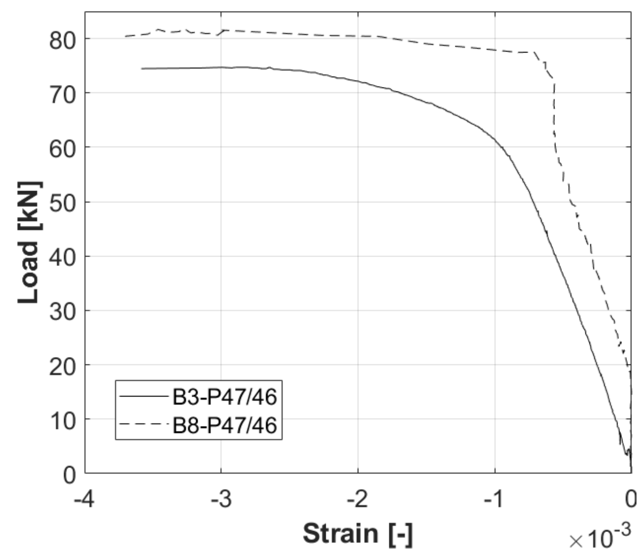

Figure 9. Load versus compressive strain.

The Figure 11 shows a comparison of the displacements recorded along the beams at 15 kN, corresponding to the elastic stage, and at three post-cracking levels, 30 kN, 55 kN and 65 kN, respectively. For the B3-P47/46 and B8-P47/46 beams, the displacements recorded up to 30 kN refer to the first loading cycle. The diagram highlights a similar behavior during the first two stages, the elastic and first-cracking phases.

The B4-P47/46 shows a significant stiffness reduction at higher load levels, which caused a considerably larger deflection. Since all these beams were removed from the same deck, homogeneous properties can be assumed for the cast-in situ slab. Accordingly, the different behavior showed under higher loads, could be related to different construction details considered for the precast PC beams. Furthermore, the preliminary non-destructive tests highlighted a worse concrete strength for the B4-P47/46 beam, confirmed by the lower recorded cracking load shown in Figure 8. However, future destructive tests will be crucial for an in-depth analysis of the global mechanical behavior. In detail, it will be necessary defining if the connectors and the roughness at the interface are effective and therefore the specimens actually behave like a composite section.

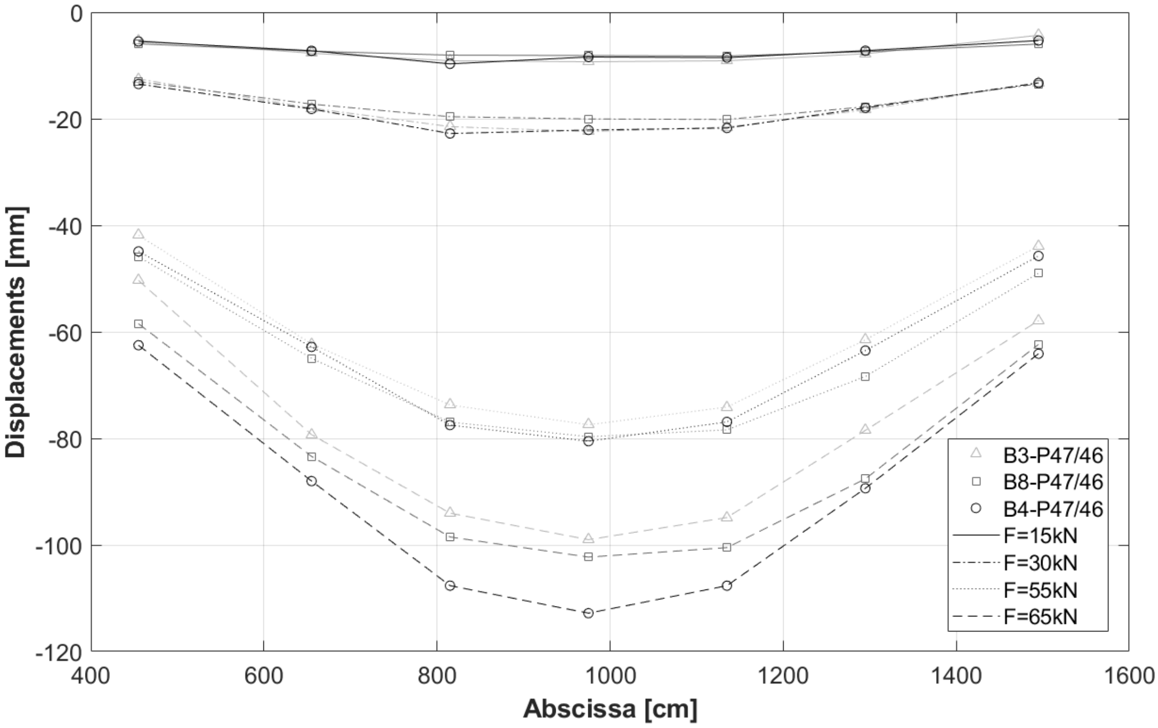

Figure 11. Displacements along the length of the beams.

5 CONCLUSIONS

This paper details the testing of three prestressed beams within the bridge|50 research project. Their behaviors at service load and at ultimate load have been explored. Some of the recorded signals were reported and commented. All beams had similar structural conditions without signs of deterioration visible from a visual inspection. The specimens failed in a brittle mode, by crushing of the top compressive fiber concrete, at ultimate load of 78 kN, 83 kN and 72 kN, respectively. Future works will concern the mechanical characterization of the materials and the loss of prestress assessment by means of destructive tests. Measured data will be compared with estimated values by existing code formulations. Furthermore, the effectiveness of the interaction between the cast-in situ slab and precast beam will be assessed in order to establish the relative contribution to the flexural stiffness and resistance of the composite section.

A further significant aspect to be investigated will be the corrosion state of the steel reinforcements. The amount of the section loss due to corrosion will be defined in order to study its influence on the bearing capacity when the volume expansion of the corrosion products is not such as to generate cover cracking. The results will be used to develop and validate a mechanical model able to reconstruct the structural behavior of existing prestressed structures.

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