Experimental tests on shear capacity of naturally corroded prestressed beams

EXPERIMENTAL TESTS ON SHEAR CAPACITY OF NATURALLY CORRODED PRESTRESSED BEAMS

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Abstract

An experimental campaign was carried out on full-scale naturally corroded prestressed concrete (PC) beams without transverse reinforcement to investigate the corrosion effects on failure modes, shear capacity and ductility. The analysed PC beams, structural members of a thermal power plant, were subjected for 10 years to refrigerating wetting cycles with marine water. In this paper, the experimental results of four-point bending tests, carried out at the Institute "Eduardo Torroja" in Madrid, are described. Before tests, a visual inspection was conducted to detect the damages induced by corrosion. During the tests, displacements and strains were measured by using Linear Variable Displacement Transducer (LVDT) and Digital Image Correlation (DIC). After the tests, strands were removed from beams and cut in pieces, which were weighed to measure the mass loss. Lastly, it was proved that the residual life of PC beams, exposed to chloride attack, is strongly affected by corrosion, whose effects reduce the shear capacity both in terms of resistance and ductility.

Keywords

Prestressed concrete beams, shear capacity, natural corrosion, Digital Image Correlation.

1. INTRODUCTION

In the last years, corrosion has been considered as one of the main factors that causes detrimental effects on durability and resistance of reinforced concrete (RC) and prestressed concrete (PC) structures and infrastructures. After the recent bridge collapses occurred worldwide, such as the Ynys-y-gwas Bridge in 1985 in Port Talbot (Woodward and Williams, 1988), the Santo Stefano Bridge in 1999 in Sicily, the Annone overpass in 2016 in Veneto, the Fossano Bridge in 2017 in Piemonte, the Morandi Bridge in 2018 in Liguria (Di Prisco, 2019), and the highway overpass in 2019 in Wuxi, extensive surveys on infrastructures were planned. Surveys outcomes revealed that many RC and PC existing structures are reaching - or already reached - the end of their service life and they show evidences of corrosion and deterioration. Research carried out in the United State by the American Society of Civil Engineers (ASCE) and the Federal Highway Administration (FHWA) report that the 9% of the USA bridges were classified as "structural deficient" and that the 30% of existing bridges have already exceeded their service life (Soltani et al.,2019). Moreover, environmental degradation can be considered the cause of more than the 7% of bridge collapses occurred from 1980 to 2012 (Di Prisco,2019). Therefore, in order to plan maintenance, repairing or dismantlement actions is fundamental to properly evaluate the effects of corrosion-induced mechanisms on RC and PC structures and infrastructures.

In case of environment-exposed structures, carbonation-induced corrosion, chloride-induced corrosion, or corrosion-induced by chloride additives into concrete can cause damage (Zhou et al., 2014). Corrosion leads to damage in concrete, such as cracking and spalling of cover caused by volume expansion, reinforcement cross-section reduction due to rust products formation and loss of mechanical performances of steel. Usually, chloride-induced corrosion can be more dangerous in prestressed members because pitting can cause damage localization and localised failures of members. Since the environmental aggressive attack - due to chloride contents or carbonatation - is increasing over the time, the capacity of corroded RC or PC existing members is gradually reducing, sometime causing unexpected failures due to lack of bending, shear or anchorage resistance. In this paper, the shear capacity reduction of prestressed reinforced concrete beams without transverse reinforcement subjected to chloride-induced corrosion is analysed.

In case of laboratory test specimens, corrosion can be naturally triggered or artificially accelerated by applying an electrical potential. In general, the artificially accelerated method produces different corrosion effects than those induced by the natural environmental attack on structures. In this paper, PC beams subjected to natural chloride-induced corrosion caused by the exposure to refrigeration wetting cycles, executed by using seawater, are analysed.

Experimental tests on corroded RC beams are available in literature. Soltani et al. (2019) pointed out that 112 specimens were tested from 1955 to 2017 to investigate the shear capacity of RC members with corroded reinforcement. However, most of the research deals with beams with transverse reinforcement. Higgins and Farrow (2006) tested nine corroded beams and five reference beams and proved that the reduction of the area of stirrups, caused by corrosion, lead to localised yielding and durability's reduction. Juarez et al. (2011) tested eight corroded and eight reference beams and demonstrated that corrosion affects both resistance and ductility of the beams. Lachemi et al. (2014) tested twelve corroded RC beam and four uncorroded beams and stated that, for low levels of corrosion - corresponding to a mass loss of about 5% - shear capacity is reduced by 5% to 25%, while for high level of corrosion - corresponding to a mass loss of about 20% - shear capacity is reduced by 50% to 75%. Rodriguez et al. (1997) tested 30 RC beams and found that for 14 corroded beams shear failure was anticipated to bending failure due to pitting at stirrups of 6 mm in diameter. Then, as soon as a stirrup failed, a diagonal crack failure occurred, as in beams without web reinforcement. Some proposals on how to deal with shear capacity of RC corroded elements (beams and slabs) are available in Contecvet Manual (2001). Contecvet Manual states that for element with web reinforcement, the shear capacity can be estimated assuming the reduced steel and concrete sections; while for elements without web reinforcement, due to the relevance of loss of bond, sometimes, the shear capacity assessment requires to move from "beam action" to "arch action" schematisation.

Despite a large amount of papers reports results of tests carried out on RC beams, only few papers provide experimental data on the shear capacity of corroded PC beams. Rinaldi et al. (2010) carried out four-point bending tests on nine artificially corroded PC beams and pointed out that corrosion affects the ultimate flexural capacity, the failure mode and finally the structural response of PC beams. Recupero et al. (2019) tested a set of prestressed beams artificially corroded. In this paper, the results of an experimental campaign, which investigates the shear capacity of corroded PC beams, are illustrated.

The few experimental papers, which are available in literature, on the shear capacity of PC beams, provide test results of artificially corroded beams with shear reinforcement. Therefore, the main novelty of this research is in the presentation of experimental test results on critical shear naturally corroded prestressed beams without transverse reinforcement. Weight loss of prestressing reinforcement reduces the reinforcement ratio, that in turn affects the shear transfer in the shear crack and in the uncracked compression zone (Marí et al., 2015; Cavagnis et al., 2018; Classen, 2020). In this paper, an experimental campaign on seven full-scale naturally corroded prestressed beams and one reference beam is presented. The experimental programme involved PC beams subjected to a natural corrosive environment for 10 years. The level of corrosion of strands was evaluated by measuring the mass loss of some pieces of strands extracted from the beams. The non-linear behaviour of PC beams, subjected to four-point bending tests, is described by plotting load vs deflection curves and measurements obtained from Digital Image Correlation (DIC).These latter measurementsare presented in terms of maximum principal strain values and deformed profiles of the PC beams for different levels of the applied load.

The results presented in this paper can contribute populating database of experimental results on corroded members. Furthermore, the presented results can be useful for the calibration of reliable analytical and numerical models capable of predicting the capacity of corroded PC beams, in terms of both resistance and ductility, and the corresponding failure mode, (Mora et al., 2018; Belletti et al., 2019; Mora et al., 2019). Indeed, in order to optimise the maintenance strategies of the existing structural heritage, design methods capable of inputting the outcomes from in-situ inspections and outputting corroded members' capacity and residual life estimation are needed.

2. EXPERIMENTAL PROGRAMME

Fourteen naturally corroded and two uncorroded PC beams were tested at the "Instituto de Ciencias de la Construcción Eduardo Torroja" in Madrid. The PC beams had the same geometric and mechanical properties and came from a refrigeration tower of a thermal power plant. During their lifecycle the PC beams were subjected to the chlorides attack induced by the refrigeration wetting cycles executed using seawater. The beams, when they were in service, were simply supported on a precast concrete beams.

Two experimental campaigns were performed from March 2018 to June 2019. During the first experimental campaign, carried out from March to June 2018, eight corroded and two uncorroded PC beams were tested, while the other six corroded PC beams were tested during the second campaign performed fromMarch toJune 2019.

In this paper, the shear capacity of seven naturally corroded PC beams and one reference uncorroded beam is investigated, while the flexural capacity of two naturally corroded PC beams and one reference uncorroded beam is illustrated in Belletti et al. (2020).

3. MATERIALS and METHOD

3.1. Design of test specimens

The full-scale prestressed beams were characterised by a rectangular cross-section having width, *b*, equal to 150 mm and height, *h*, equal to 300 mm.

The prestressing reinforcement consisted of two seven wires strands 1/2 S having an equivalent diameter equal to 12.9 mm, placed at the bottom of the cross-section. The yield strength, $f_{p0,1k}$, was equal to 1580 MPa, while the nominal ultimate strain, ε_u, was equal to 5%. An initial prestressing strength, σ_p, equal to 1408 MPa was applied to PC beams. The top longitudinal reinforcements consisted of two Φ5 steel ribbed bars. The beams were built without transverse reinforcement and the distance of the centroid of the reinforcement from the outer surface was equal to 40 and 50 mm, respectively, for the reinforcement placed at the top and at the bottom of the cross-section of the beam. Figure 1 shows the beams geometry and reinforcement details. Table 1 reports the mechanical properties of the steel, as indicated in the design drawings of the beams.

Cylindrical concrete compressive strength, f_{cm}, was estimated from destructive compressive tests performed on cores extracted - after failure - from specimens PB4P9, PB4P13 and PB4P14. From each beam, four cylindrical samples 100 mm high and with a diameter equal to 50 mm were extracted. Compressive uni-axial tests were performed by using an Automax 5 machine, working at a load rate equal to 0.6 MPa/s. Finally, a mean value of concrete compressive strength equal to 45.4 MPa was obtained.

Before tests, several damages due to corrosion of strands were visible, such as concrete cover spalling in proximity to the beams' ends, longitudinal cracks, and swellings, as shown in Figure 2.

3.2. Test Set-up

All the specimens were subjected to load-control four-point bending tests, which were carried out up to failure by means of a servo-hydraulic actuator with a nominal maximum capacity equal to 200 kN, as shown in Figure 3. The PC beams were simply supported at both ends. The distance between supports, *L*, is varying depending on beams' damage at their ends. Two symmetrical loads were applied by means of a steel beam at a distance equal to *L*/3 from supports, Figure 3(a). In order to measure the deflection of the beam, a Linear Variable Displacement Transducer (LVDT) was placed at the bottom of the beam at mid-span, as shown in Figure 3(a). The instrumentation was completed with steel plates having dimension 70 mm x 200 mm x 8 mm located in correspondence of supports and loading points' positions.

The loading points were not symmetric with respect to the beam length because the distances between supports and beam ends, d_1 and d_2, were not equal due to corrosion damage. Indeed, the supports were placed 200 mm far from the corner or, in case of spalling of concrete cover or longitudinal cracks, 200 mm far from the end of the damage. Table 2 summarizes the main geometrical dimensions of the tested PC beam.

An identifying code, composed by letters and numbers, is used to name the beams: PB means prestressed beam and 4P means four-point bending test, Table 2. Furthermore, the letter N is added after the abbreviation PB to denote the uncorroded specimen (reference beam). Table 2 reports, for each PC beams, the details of the loading setup and the shear span-to-depth ratios, *a/d*, - being the shear span, *a*, equal to *L/3*-.

In this paper, DIC methodology was used to monitor the non-linear behaviour of PC beams during loading up to failure. A high contrast speckle pattern was applied to the beams' surfaces in a region of interest (ROI) delimited by supports, Figure 3(a), and constant illumination was provided in order to avoid interferences caused by the shadow effect. The high-resolution digital camera Nikon D7200 completed the DIC system (24.72 million pixels). Images were post-processed using the open source two-dimension software package Ncorr (Schreier et al., 2009; Tambusay et al., 2018; Blaber et al., 2015), which works in MATLAB environment. The region of interest (ROI) was 300 mm high and with a variable length depending on the distance between supports, as shown in Figure 3(a). The post-processing phase consisted in the comparison of the reference image representing the undeformed shape of the beam - captured before the application of the load - and the images representing the deformed shape of the beam - captured every 5 kN load increment. Displacements and strains values were obtained by overlapping the images of undeformed and deformed beams. In this paper, the contour plot of maximum principal strain values and the deformed beams' profiles for different values of applied load are reported.

3.3. Corrosion Damage Detection

3.3.1. **Crack pattern maps**

Before the four-point bending tests, visual inspections were performed to detect areas damaged by corrosion, which may be potential zones where brittle failures may occur. Figure 5(a) - Figure 11(a) show longitudinal cracks, spalling of concrete cover at beams' ends, and swelling, drawn in their exact location and coloured in red in the crack pattern maps. The crack pattern map of the uncorroded beam PBN4P2 (reference beam) is not reported because no damage was visible on the beam surface before the test. Figure 4 shows the photo of the beam PBN4P2 at failure.

Before tests, spalling of concrete cover at the left end of the beam was visible for PB4P5 and PB4P8 beams, as shown in Figure 5(a) and Figure 8(a). PB4P8 beam presented swelling in correspondence of the right end of the beam.

Spalling of concrete cover at both ends was visible for PB4P6 and PB4P7 beams, as shown in Figure 6(a) and Figure 7(a). Furthermore, the crack pattern map of PB4P7 beam shows branched cracks at mid-height in proximity of the right end of the beam, Figure 7(a).

PB4P9 beam's crack map shows a longitudinal crack at the left beam's end in correspondence of strands' depth due to splitting phenomena and spalling of concrete cover at the right end of the beam, Figure 9(a).

The crack map of the PB4P13 beam shows longitudinal cracks at both beam ends in correspondence of strands' depth and swelling at the top of the beam which extended from the loading point to the right end of the beam, Figure 10(a).

PB4P14 beam's crack map shows a longitudinal crack at the left beam end in close proximity to the cross-section mid-depth, Figure 11(a).

3.3.2. **Corrosion Level Detection**

After the four-point bending tests, the prestressing strands were removed from the beams and cut in pieces, 500 mm long. The mass loss was measured by weighing the pieces after cleaning rust products. The cleaning was executed by using a 37% hydrochloric acid solution and a tank instrument with ultrasound energy. The mass loss, η_s, was calculated according to Eq.(1):

$$\eta_s = \frac{m_0 - m}{m_0} \tag{1}$$

where m_0 is the mass per unit length of the uncorroded strand's piece and m is the final mass per unit length of the strand's piece after cleaning.

Table 3 reports the three different levels of corrosion, which correspond to three value ranges of measured mass loss. Level LVI - coloured in blue - corresponds to a low level of corrosion, level LVII - coloured in green - corresponds to a medium level of corrosion, while level LVIII - coloured in red - corresponds to high level of corrosion. The same colours in Figure 5(b) - Figure 11(b) indicate the strands' level of corrosion along the length of PC beams.

4. RESULTS and DISCUSSION

In general, the shear resistance of PC beams without shear reinforcement and with corroded strands is affected by the reduction of the transversal area of strands, the prestressing force and the bond between concrete and steel.

Two types of failure were experimentally observed at the end of the four-point bending tests, in the following denoted as Shear I and Shear II.

Shear I failure mode was achieved after the formation of bending vertical cracks, perpendicular to the axis of the beam, which developed at the bottom of the beam between the loading points where the constant value of bending moment was applied. Afterward, shear diagonal cracks developed in the shear spans, in the regions of constant shear force, from the bottom to the top of the beam. Finally, when cracks reached the loading steel plate position, at the top of the beam, the beam collapsed, as shown in Figure 12(a).

Shear II failure mode was achieved after the formation of a single shear diagonal crack, that suddenly developed in correspondence of the more damaged shear spans of the beam; in this case bending vertical cracks were not visible, as shown in Figure 12(b).

Shear I failure mode was more ductile than Shear II failure mode. Shear I failure mode could be anticipated by the yielding of the strands, but flexural failures did not occur during the tests. According with the most recent approaches adopted for the description of the mechanisms involved in the shear resistance of members without transverse reinforcement (fib, 2013), the Shear I resistance of beams characterised by the same shear span is decreasing as the corrosion level is increasing. Indeed, the corrosion of strands causes the reduction of both transversal area of strands and prestressing force, which increases strains at mid-section and descreases aggregate interlocking.

Both failure modes Shear I and Shear II occurred independently on shear span-to-depth ratios. Anyway, interaction between shear and bending can be recognised as the shear span-to-depth ratio was varying, (Leonhardt, 1965). In general, localised high levels of corrosion causes strain concentration and embrittlement of the global response of the beam (Sanchez et al., 2017a; Sanchez et al., 2017b).

PBN4P2 uncorroded beam failed according to Shear I mode; from experimental outcomes of three points bending tests, reported in Belletti et al. (2020), result that the flexural capacity of the uncorroded PC beams was equal to 88 kNm. Since the interaction between shear force and bending moment is relevant in all the analysed PC beams, Table 4 reports the values of the peak load, the maximum shear force, the bending moment, and the failure mode. PB4P5, PB4P6, PB4P8, PB4P13, and PB4P14 beams failed according to Shear I mode after an initial development of vertical bending cracks, diagonal cracks occurred in correspondence of shear spans, as shown in Figure 4, Figure 5(c), Figure 6(c), Figure 8(c), Figure 10(c), and Figure 11(c). Figure 4 shows that PBN4P2 beam's crack pattern at failure was almost symmetric with respect to the mid-span of the beam. On the contrary, a more pronounced diagonal crack, which lead to the failure, developed in correspondence of the shear span affected by the highest level of strands' corrosion in PB4P5, PB4P6, PB4P8, PB4P13 and PB4P14 beams.

PB4P7 and PB4P9 beams failed according to Shear II mode after a sudden formation of a single shear diagonal crack, as reported in Figure 7(c) and Figure 9(c). PB4P7 beam failed on the side affected by the highest level of corrosion; on the contrary PB4P9 beam failed on the opposite side. The crack patterns of PB4P7 and PB4P9 beams show the formation of an extended horizontal crack at the strand's depth that may be caused by bond failure. Indeed, it is well known that Kani's tooth model requires that bending moment, which is produced by bond force in strands, is in equilibrium with bending resistance of the cantilevered teeth, (Kani, 1966); Collins and Mitchell, 1991). Strand's debonding could be caused by splitting cracking induced by dowel action mechanisms, which were activated for shear transfer. Indeed, in beams without transverse reinforcement, dowel action mechanism is limited by tensile strength of concrete, (Park et al., 1975). Figure 7(c) and Figure 9(c) show that failure was suddenly achieved after the formation of splitting cracks at strand's depth.

Shear I and Shear II failure modes depend not only on shear span-to-depth ratios, which were varying for the tested beams, but also on the position and degree of strands' corrosion.

In general, for uncorroded RC beams there are two combined mechanisms to resist shear: beam action and arch action. Beam action develops with variable tensile force at reinforcement (bond is needed) and almost constant lever arm - if the beam has not stirrups, concrete tensile strength at the web is relevant. Arch action develops with variable lever arm and constant tensile force at reinforcement (no bond) but an anchorage at the end of the beam would be needed to bear the tie action in the arch. Experimental tests carried out by Leonhardt (1965), on RC beams without web reinforcement and no damage, demonstrated that shear failures depend on shear span-to-depth ratios. For shear span-to-depth ratios ranging from 3 to 7, beam action failure occurs at, or shortly after, the application of the diagonal cracking load; for shear span-to-depth ratios ranging from 2 to 3, arch action failure occurs due to shear compression or flexural tension failure of the compression zone above diagonal cracking load; for shear span-to-depth ratios lower than 2.5, arch action failure occurs due to crushing/splitting of concrete. Thus, for shear span-to-depth ratios below 7, the flexural capacity of the beams is not attained and shear governs the design (Park et al., 1975). The ranges of shear span-to-depth ratios adopted for RC beams without web reinforcement cannot be directly used to schematise the failure mode of the PC beams presented in this paper for several reasons. Firstly, the prestressing force applied at a determinate eccentricity with respect to the center of the cross section generates additional arch action mechanisms. Secondly, corrosion damage caused strain localisation that enables to easily schematise the resisting mechanisms; indeed, resisting mechanisms can change or can reduce/increase their effects along the beam length. Thirdly, loss of bond might have occurred, causing the reduction of beam action and arch actions mechanisms. Finally, the decrease of compression concrete stresses, due to reduction of prestressed force, has happened, weakening the shear performance.

4.1. Load-Deflection curves

Figure 13 shows experimental load-deflection curves of tested PC beams. Figure 13(a) shows that the ductility of PBN4P2 uncorroded PC beam is much higher than the ductility of corroded beams, not only because this beam has the highest shear span-to-depth ratio (equal to 6.7) but also because stress localisation due to corrosion effects did not occur. The ultimate mid-span deflection was equal to 120.70 mm, while the peak load was equal to 98.47 kN.

A stiffer elastic response was expected for beams PB4P5 - PB4P9 because the shear span-to-depth was ranging between 5.2 and 5.7. In reality, the strong reduction in corroded beams' ductility and resistance is due not only to the lower shear span-to-depth ratios but also to corrosion effects. Figure 13(b)-(f) show that the beneficial effect of prestressing on shear capacity of beams is strongly compromised by strands' corrosion. Furthermore, the reduced tensile resistance of strands caused a more pronounced interaction between flexural and shear mechanisms. Indeed, where strands were corroded, higher flexural crack opening widths developed causing detrimental effects on aggregate interlock mechanism, (Park et al., 1975).

Figure 13(g)-(h) show that load-deflection curves of PB4P13 and PB4P14 beams reached the highest shear capacities and the lowest ultimate displacements because of the lowest shear span-to-depth ratios. Indeed, according to Kani's comb model, cracked beams, having low values of shear span-to-depth ratios, can form a tied arch that can support loads even after the failure of cantilevered teeth, (Collins and Mitchell, 1991). PB4P5 and PB4P6 beams, both having shear span-to-depth ratios equal to 5.7, failed in Shear I mode in correspondence of similar values of peak load - respectively equal to 90.21 kN and 89.39 KN - and ultimate displacement at mid-span - respectively equal to 35.20 mm and 34.06 mm, Figure 13(b), (c). The higher level of corrosion of PB4P6 beam caused a slightly lower resistance and ductility with respect to PB4P5 beam. The shear span-to-depth ratios of PB4P7 and PB4P9 beams - equal to 5.5 and 5.4, respectively - were lower than shear span-to-depth ratios of PB4P5 and PB4P6 beams. Therefore, lower ultimate displacement values of PB4P7 and PB4P9 beams - respectively equal to 29.66 mm and 27.88 mm - than PB4P5 and PB4P6 beams were expected, Figure 13(d), (f). On the contrary, the lower peak load values of PB4P7 and PB4P9 beams - respectively equal to 81.74 kN and 80.00 kN - than PB4P5 and PB4P6 beams could be caused by corrosion damage. Indeed, PB4P7 beam failed in correspondence of the shear span affected by the highest level of strands' corrosion, while PB4P9 beam failed due to debonding on the side of the beam where spalling of concrete cover was observed, Figure 13(d), (f).

PB4P8 beam, having a shear span-to-depth ratio equal to 5.2, failed in correspondence of a higher value of the peak load - equal to 100.06 kN - than PB4P7 beam and a lower value of ultimate displacement - equal to 28.69 mm, Figure 13(e).

PB4P13 and PB4P14 beams, having the lowest values of shear span-to-depth ratios - respectively equal to 4.3 and 4.1 - failed in correspondence of the highest values of peak load - respectively equal to 127.45 kN and 145.9 kN - and the lowest values of ultimate displacement - respectively equal to 23.7 mm and 22.3 mm, Figure 13(g), (h).

4.2. Principal strains field obtained from DIC

Additional data on the principal strains field were obtained by performing DIC analyses after the experimental test. The maximum principal strain values obtained using the DIC analyses were in good agreement with the crack patterns observed during the tests. In Figure 13, the load-deflection curves are marked to identify the load values corresponding to the contour plots of the maximum principal strain illustrated in Figure 14 - Figure 21. It is important to note that the contours' legends illustrated in Figure 14 - Figure 21 were not scaled by using the same maximum value and the same number of intervals for all the beams. Anyway, for each beam, the same legend is used to appreciate the crack pattern development. Figure 14 - Figure 21 confirmed that shear diagonal cracks developed always in correspondence of the shear span affected by the highest level of corrosion detected along the strands, with the sole exception of PB4P9 beam who failed probably due to strands' debonding, Figure 19.

Figure 14 - Figure 16, Figure 18 and Figure 20 - Figure 21 show the maximum principal strains measured from photos of PBN4P2 - PBN4P6, PB4P8 beams and PB4P13 - PB4P14, revealing a Shear I failure mode . These latter images reveal the presence of vertical bending cracks located in the central part of the beam, where the bending moment its constant and reached the maximum value. As the load was increasing, the vertical bending cracks increased their opening width values. Figure 14 and Figure 18 show the last contour plots, recorded for an applied load equal to 95 kN and 100 kN, for PBN4P2 and PB4P8 beams respectively. PBN4P2 and PB4P8 beams developed two shear diagonal cracks, which are approximately symmetric with respect to the mid-span.

Figure 15 and Figure 16 show the principal strains evolution recorded from photos of PB4P6 and PB4P7, respectively. The first images of the failure sequence highlight the presence of a vertical bending cracks in correspondence of the bottom of the beam - when the applied load was equal to 75 kN. The last recorded images, corresponding to a load equal to 90 kN and 85 kN respectively, reveal the formation of a wide shear diagonal crack at the right and left side, respectively, where the highest levels of corrosion were found.
Figure 17 and Figure 19 show the principal strains evolution recorded from photos of PB4P7 and PB4P9 beams. During the tests, vertical bending cracks were not visible, while shear diagonal cracks were recorded for the first time when the applied load was equal to 60 kN and 70 kN, for PB4P7 and PB4P9 beams, respectively. The last recorded images of PB4P7 and PB4P9 beams, were taken in correspondence of an applied load equal to 80 kN and 75 kN, respectively. Figure 17 and Figure 19 show the formation of a single shear diagonal crack.In PB4P7 beam the diagonal crack develops in correspondence of the higher level of strands' corrosion, while the failure of the beam PB4P9 occurred at the opposite side.
Figure 19 shows that for PB4P9 beam, whose failure was strongly affected by strands' debonding. The distance between supports and the first crack is lower than for other beams, meaning that prestressing force value is lower in PB4P9 beam than in other beams. PB4P9 beam unexpectedly failed in the shear span - on the right side of the beam - were corrosion was not detected but spalling of concrete cover was observed on the right end of the beam which could cause detrimental effects on bond resistance.
Figure 20 and Figure 21 show the contour plots of maximum principal strains measured in PB4P13 and PB4P14 beams, respectively. Figure 20 and Figure 21 show the formation of vertical cracks, which were not visible to the naked eye during the experimental tests.

4.3. Deformed shape of beams obtained from DIC

Vertical displacements, obtained from DIC analyses, were useful to plot, the deformed shape of the beams along their lengths in correspondence of different values of applied load, Figure 22. In general, the deformed shape showed a parabolic trend, symmetric with respect to the mid-span of the beam where the maximum value of vertical displacement was recorded. Similar profiles were obtained for all the tested specimens, except for PB4P5, PB4P7, and PB4P9 beams. Indeed, PB4P5, PB4P7, and PB4P9 beams were characterised by an un-symmetric profile of the deformed shape and a strong displacement localisation where shear diagonal cracks occurred. This discontinuity of the beams' deformed shape could be due to bond failure. Indeed horizontal cracks, which formed at strand's depth, were visible to the naked eye in PB4P5, PB4P7 and PB4P9 beams, Figure 5, Figure 7, and Figure 9. Figure 22(b), Figure 22(d), and Figure 22(f) show that, where splitting horizontal cracks developed - in PB4P5, PB4P7, and PB4P9 beams - a strong increase of vertical displacements was observed. The localised increment of vertical displacement values could be due to a localised strong stiffness reduction caused by strands' debonding. Strand's debonding could be caused by a mix of reasons affecting the shear transfer, such as splitting of concrete induced by rust volume expansion, reduction of tensile force in strands induced by mass loss or dowel action effects on concrete cover.

5. CONCLUSIONS

In this paper, the results of experimental tests on seven naturally corroded PC beams and one uncorroded control PC beam are presented and discussed. In order to investigate the shear capacity dependency on corrosion effects, four-point bending tests were carried out up to failure. Finally, the main outcomes from visual inspection, as crack pattern maps and mass loss weighing, have been correlated to observed shear failures and measurements of displacements.

According to the results obtained from the experimental tests, the following conclusions are listed:

- In general, the beams were characterised by different shear span-to-depth ratios, therefore results observed for the uncorroded PC beam cannot be directly compared with the results obtained for the corroded PC beams.
- The strands' corrosion caused strain localisations and premature failure modes. Shear resistance of beams characterised by similar shear span-to-depth ratios decreased as the corrosion level of the strands increased.
- The beneficial effect of prestressing on shear capacity of beams was strongly compromised by strands' corrosion. Indeed, the reduced tensile resistance of corroded strands caused a more pronounced interaction between flexural and shear mechanisms.
- The zones of the PC beams affected by higher level of corrosion were potential zones where higher diagonal crack opening widths developed causing detrimental effects on aggregate interlock mechanism.
- Lastly, this experimental campaign demonstrates that the knowledge of the corrosion level distribution over the length of PC members is fundamental for a reliable prediction of the effects of strain localisation on the non-linear behaviour of structures and infrastructures. Indeed, well established mechanisms, such as beam action or arch action, which are used for schematise the shear transfer of RC beams, need to be modified in order to consider the localisation effects produced by corrosion in PC beams.

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List of Tables

Table 1 Steel mechanical properties.

Type of reinforcements	Diameter [mm]	Yield strength [MPa]	Modulus of elasticity [GPa]	Ultimate strain [%]
Rebars	5	435	195	18
Strands	12.9	1580	195	5

Table 2 Details of the loading setup.

Specimens	**Configuration**	**L [mm]**	**d_1 [mm]**	**d_2 [mm]**	**a/d**
PBN4P2	4 Point Bending	5040	200	200	6,7
PB4P5	4 Point Bending	4260	980	200	5,7
PB4P6	4 Point Bending	4240	200	1000	5,7
PB4P7	4 Point Bending	4100	200	1140	5,5
PB4P8	4 Point Bending	3900	1340	200	5,2
PB4P9	4 Point Bending	4060	950	200	5,4
PB4P13	4 Point Bending	3210	1000	1000	4,3
PB4P14	4 Point Bending	3110	1100	1000	4,1

Table 3 Corrosion levels, mass loss, and strand samples.

Corrosion Level	Corrosion pattern colour	Mass loss	Samples
LVI	Blue	$\eta_s < 2$ %	
LVII	Green	$2 < \eta_s < 10$ %	
LVIII	Red	$\eta_s > 10$ %	

Table 4 Main experimental results: peak load, shear, moment, and failure mode.

Beam	a/d	Peak load [kN]	Shear force [kN]	Bending moment [kNm]	Failure mode
PBN4P2	6,7	98.47	49.24	82.71	Shear I
PB4P5	5,7	90.21	45.11	64.05	Shear I
PB4P6	5,7	89.39	44.70	63.17	Shear I
PB4P7	5,5	81.74	40.87	55.86	Shear II
PB4P8	5,2	100.06	50.03	65.04	Shear I

PB4P9	5,4	80.00	40.00	54.13	Shear II
PB4P13	4,3	127.45	63.73	68.19	Shear I
PB4P14	4,1	145.90	72.95	75.62	Shear I

Figure captions

Figure 1.Cross-section of the beams and longitudinal reinforcements.

Figure 2. Damage induced by corrosion of strands: (a) strands' corrosion, (b) spalling of concrete cover, (c) swelling, and (d) longitudinal crack.

Figure 3. Test setup configuration: (a) four-point bending and region of interest (ROI), and (b) hydraulic actuator.

Figure 4. PBN4P2 beam at failure.

Figure 5. Beam PB4P5: (a) crack pattern map before the test, (b) corrosion level pattern, and (c) photo at failure.

Figure 6. Beam PB4P6: (a) crack pattern map before the test, (b) corrosion level pattern, and (c) photo at failure.

Figure 7. Beam PB4P7: (a) crack pattern map before the test, (b) corrosion level pattern, and (c) photo at failure.

Figure 8. Beam PB4P8: (a) crack pattern map before the test, (b) corrosion level pattern, and (c) photo at failure.

Figure 9. Beam PB4P9: (a) crack pattern map before the test, (b) corrosion level pattern, and (c) photo at failure.

Figure 10. Beam PB4P13: (a) crack pattern map before the test, (b) corrosion level pattern, and (c) photo at failure.

Figure 11. Beam PB4P14: (a) crack pattern map before the test, (b) corrosion level pattern, and (c) photo at failure.

Figure 12. Shear failure modes: (a) Shear I failure mode - PB4P5 beam - and (b) Shear II failure mode - PB4P7 beam.

Figure 13. Load-deflection curves: (a) PBN4P2, (b) PB4P5 and (c) PB4P6, (d) PB4P7, (e) PB4P8, (f) PB4P9, (g) PB4P13, and (h) PB4P14.

Figure 14. DIC of beam PBN4P2.

Figure 15. DIC of beam PB4P5.

Figure 16. DIC of beam PB4P6.

Figure 17. DIC of beam PB4P7.

Figure 18. DIC of beam PB4P8.

Figure 19. DIC of beam PB4P9.

Figure 20. DIC of beam PB4P13.

Figure 21. DIC of beam PB4P14.

Figure 22. Vertical displacement along the beam length for different load values: (a) PBN4P2, (b) PB4P5, (c) PB4P6, (d) PB4P7, (e) PB4P8, (f) PB4P9, (g) PB4P13, and (h) PB4P14.