Seismic strengthening of RC beams using post-tensioning with anchorages by bonding

Reforço sísmico de vigas de betão armado por aplicação de pós-tensão com ancoragens por aderência

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Abstract

Moment resisting frames are one of the most common structural systems in RC buildings. This study is focused on the behaviour of beam plastic hinges and a strengthening solution using internal bonded post-tensioning strands. Two specimens have been tested in this experimental campaign, reference specimen CB1 and strengthened specimen CB2. The loading history was a combination of cyclic loading and the gravity load. Two internal pretensioned parallel post-tensioning strands were installed in specimen CB2 at one third of the span length close to the end support. Epoxy resin was injected in the extremities of the prestressing strands to guarantee the anchorages and the length of the strands between anchorages was left unbonded to assure uniform stress. The strengthened beam exhibited enhanced cyclic behaviour, with increased load capacity and decreased residual deformations. The energy dissipation for drift 3.5 % was increased significantly when compared to the reference specimen.

Resumo

Os sistemas porticados são soluções estruturais frequentemente adotadas em edifícios de betão armado. Este estudo incide no comportamento de rótulas plásticas e respetiva solução de reforço por aplicação de pós-tensão com ancoragens por aderência. A campanha experimental compreendeu o ensaio de dois modelos: de referência CB1 e reforçado CB2. Os modelos foram submetidos a uma história de carregamentos resultante da imposição de deslocamentos cíclicos em simultâneo com carga gravítica. O modelo CB2 foi reforçado com dois cordões de pré-esforço, localizados a terços de vão junto do apoio. A ancoragem por aderência dos cordões foi conseguida através da injeção de resina epoxídica nas extremidades, permitindo garantir que o comprimento não-aderente entre ancoragens apresente tensões uniformes. A solução reforçada exibiu um comportamento cíclico melhorado, aumento da capacidade de carga e decréscimo das deformações residuais. A energia dissipada para um *drift* de 3,5 % aumentou significativamente em relação ao modelo de referência.

Keywords: RC beam plastic hinge / Cyclic loading / Gravity load / Prestressing steel strands / Bonding

Palavras-chave: Rótulas plásticas de vigas de betão armado / Carregamentos cíclicos / / Carga gravítica / Cordões de aço de pré-esforço / Aderência

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1 Introduction

In Eurocode 1998-1 [1], critical regions detail depends on the level of ductility required in the design. Further recent studies in New Zealand indicate that the rotation and longitudinal reinforcement strain observed in plastic hinges regions in the ultimate limit state do not only depend on ductility factor [2]. Accordingly, the New Zealand Code [3] includes additional factors that influence the behaviour of plastic regions under seismic actions. It classifies beam plastic hinges into two main groups, unidirectional and reversing. The formation of reversing plastic hinges, among other factors, depends on the gravity load level, more specifically, on the ratio between the shear induced by the gravity loads and the shear induced by the seismic action. If this ratio is low, reversing plastic hinges form. When the gravity load has considerable values, this ratio is higher and unidirectional hinges are formed. In the behaviour of unidirectional hinges, the relationship between inelastic rotation and inter-story drift is not direct due to the elongation of the beam [4] and accumulation of deflections [5]. On the other hand, the behaviour of reversing hinges has been thoroughly studied, and its rotation depends on the inter-story drift. Preventing the formation of unidirectional hinges is a desirable solution. This was firstly proposed by Fenwick *et al.* [5], [6] and now it is part of the New Zealand Code NZC 1170.5 [3]. Consequently, in the present research work, gravity load effect is taken into consideration in the cyclic loading tests, aiming to predict correctly the failure mechanisms developed in structures during an earthquake.

Few researchers have taken into account the effects of gravity loads on beams besides the quasi-static cyclic loading (Walker *et al.* [7], Dhakal *et al.* [2], Proença et al. [8] and Gião *et al.* [9], [10], [11]). Walker *et al.* [7] used 75% of the theoretical flexural strength as gravity loading in a RC beam and observed both reversing and unidirectional plastic hinges. Dhakal *et al.* [2] proceed to the evaluation of material strain in plastic hinge regions based on their calculated curvatures and the level of ductility that they are designed to sustain. Proença *et al.* [8] applied a combined cyclic and gravity load procedure in steel beam to column connections. The failure modes of conventional monotonic and the proposed loading procedure were compared. Two gravity load percentages were studied, corresponding to 25% and 75% of the designed moment. Gião *et al.* [10], [11] followed similar loading history in reinforced concrete beam to column specimens, but the gravity load was chosen as the half of the yielding moment. This test procedure involved a combination of forced controlled test for the imposition of the gravity load and displacement-controlled test for the cyclic displacement. Three specimens were tested according to this protocol by Gião *et al.* [9], one control specimen and two strengthened ones, and all of them failed in the compression zone.

Post-installed anchorages have been the focus of many researchers and their behaviour has been exploited in various conditions, loading histories, etc. Nevertheless, in almost all cases, these anchors usually involve threaded rods or other similar anchors. Faria *et al.* [12] investigated bonding behaviour of prestressing strands for various embedment lengths. Furthermore, this solution was applied as a strengthening technique [13] for improving the punching shear behaviour of flat slabs.

Mimoto *et al.* [14] also investigated strengthening of beams by internal post-tensioning tendons. Gião *et al.* [11] studied strengthening of RC beams by external post-tensioning subjected to cyclic and gravity load [9]. The strengthened specimen had an enhanced behaviour, dissipated more energy and reduced accumulated deformation was observed.

The experimental cyclic loading tests of two beam to column specimens subjected to gravity and cyclic loading history are described in this paper. Specimen CB1 [15] represents the reference specimen and specimen CB2 represents the strengthened specimen. The objective of this study is the seismic strengthening of RC beams by prestressing bonded steel strands, similarly to Faria *et al.* [13] but applied on RC beams. In the present study, a prestressing force of 200 kN was applied in the beam through two prestressing steel strands (100 kN in each strand) followed by injection of the epoxy resin to ensure strand anchorages in its extremities. Specimen CB2 behaviour was compared with the reference specimen CB1 in terms of maximum load, accumulated deformations and dissipated energy.

2 Experimental program

2.1 Specimens

Two specimens were studied in this campaign, named respectively CB1 [15] and CB2 (C stands for Cyclic and B for Beam). Both specimens had the same geometry, reinforcement details and cross section, as given in Figure 1.

The specimen geometry represents one third of the clear span and the given cross section corresponds to the beams end support. The aim of this research is to enhance the seismic behaviour of the beam; thus, the column was built as a rigid block and beam-to-column joint behaviour has been neglected. Specimen CB1 represents the reference specimen and specimen CB2 represents the strengthened specimen, Figure 2.

Material characteristics are summarized in Table 1. Concrete mechanical characteristics were experimentally evaluated. Compressive strength was estimated as the average of three (CB1)

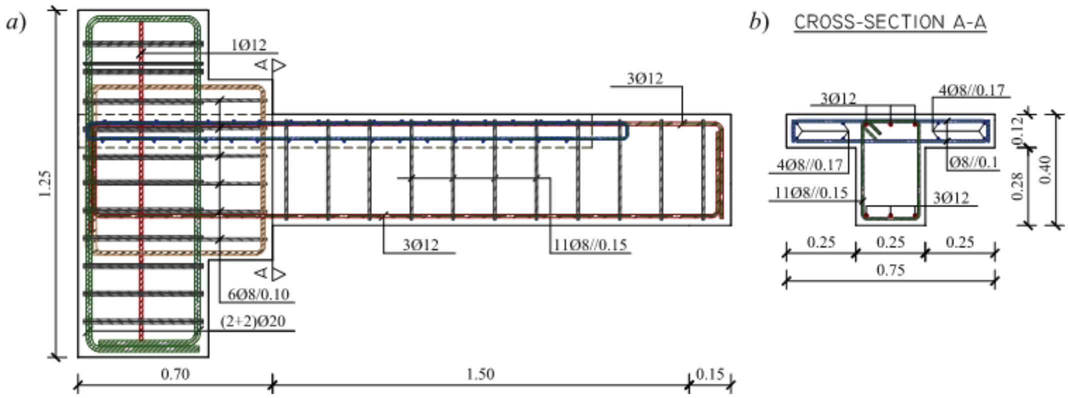


Figure 1 Specimens CB1 and CB2 geometry: a) elevation and b) cross section

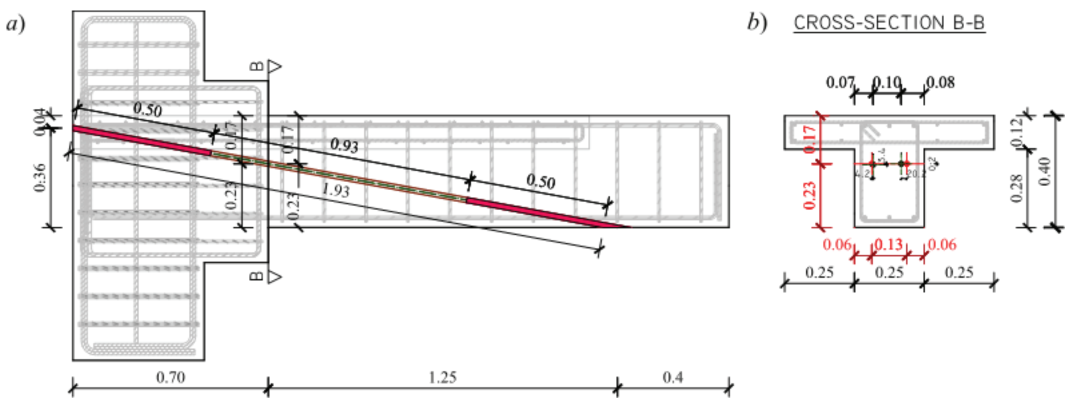


Figure 2 Strengthened specimen CB2: a) strands layout – profile and b) strands in the cross section

to four (CB2) cylinder compression tests. Tensile strength was evaluated from tensile splitting test of three (CB1) to six (CB2) cylinders, the given value represents the 90 % of the tensile splitting test experimental value, as recommended in Eurocode 1992-1-1 [16]. Elasticity modulus was obtained experimentally from four (CB2) to five (CB1) cylinders. Steel properties were determined experimentally, excluding modulus of Elasticity which was accepted 200 GPa, as the mean values obtained in the tensile test of five samples for each bar diameter. The values given in Table 1 correspond to the longitudinal bars of diameter ϕ 12 for both specimens.

Table 1 Specimens CB1 and CB2 – Mechanical properties of the materials

Specimen	Concrete characteristics			Steel characteristics				
	f_{cm} (MPa)	f_{ctm} (MPa)	E (GPa)	f_y (MPa)	ε_y (%)	ε_{ult} (%)	f_{ult} (MPa)	E (GPa)
CB1	25.20	2.26	28.32	541.50	0.27	11.73	630.40	200.00
CB2	37.94	2.80	33.15	515.64	0.26	12.27	618.87	200.00

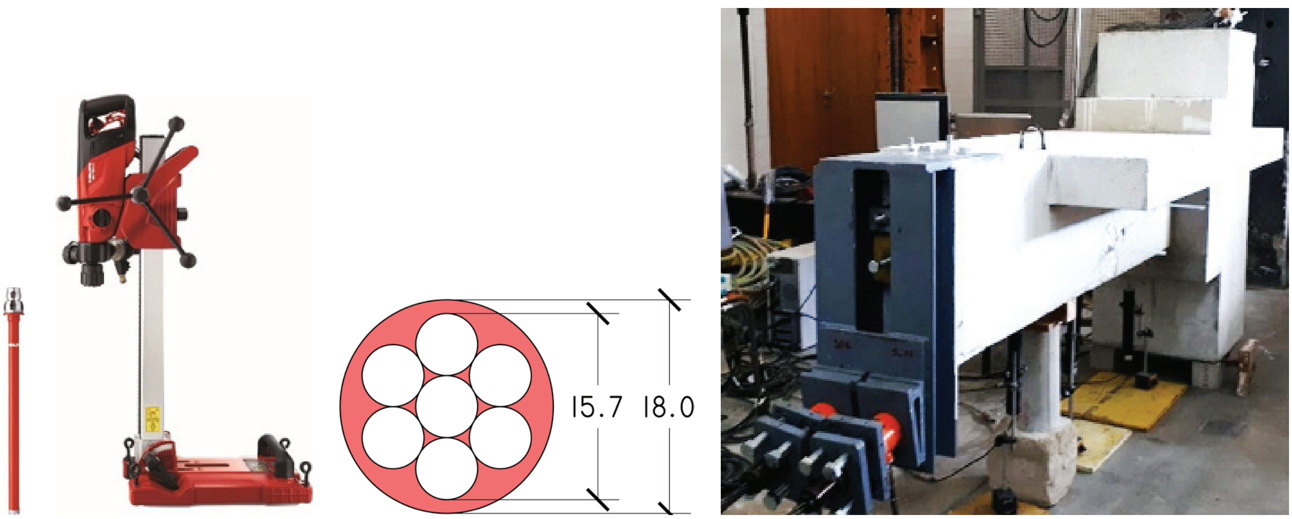
Specimen CB2 drilling geometry and strands profile, shown in Figure 2, were driven by many factors, such as: clear span length of the beam, beam cross section, reinforcement details, prestressing steel strand characteristics, steel strand bonded and unbonded lengths, bond stress-slip law of bonded anchorage, etc. Anchorages were placed in one third of span and on beam-to-column joint region, due to owing less probability for development of cracks in these regions. Bonded anchorages placed in cracked concrete have a lower bearing capacity and are unsafe [17] for seismic design when compared with anchorages in uncracked concrete. The strands geometry was defined by combining specimen geometry limitations with prestressing steel strand characteristics (load bearing capacity and its respective strain).

Push-in and pull-out after push-in tests of bonded prestressing strands have been carried out previously to define the anchorage length associated with the respective load capacity. The prestressing force applied in steel strands was 100 kN, as they were designed to remain within elastic behaviour up to their load bearing capacity for the designed drift beam rotation of 3.5%. The anchorage length used in specimen CB2 was of 500 mm, as presented in Figure 2.

HILTI diamond drilling system was used for the execution of hollow deep drilling holes. This system consisted of a drilling machine (Figure 3b) that allows drilling with specific angles (DD 150-U), various diamond drilling bits (Figure 3a) of 18 mm diameter with different lengths (320, 1000, 1750 and 2500 mm).

Two prestressing steel strands of 15.7 mm nominal diameter (Figure 3c) and cross section area of 150 mm^2 were placed in the previously 18 mm drilled holes of specimen CB2, as shown in. These strands were chosen due to their ability to undergo high stress rates and reduced cross section dimension that allow installation in relatively small sections. The prestressing steel has a modulus of elasticity of 195 GPa, and ultimate and proof force at 0.1 % were, respectively, 286 kN and 246 kN. Before placement, and in order to improve adhesion, strands were cleaned with wire-brush and solvent for removing dust or impurities during transportation and storage time.

The unbonded length was wrapped by a plastic adhesive tape. Temporary mechanical anchorages were used for applying a prestressing force of 100 kN in each of the prestressing strands. Then, the epoxy resin HIT-RE 500 V3 was injected in strand extremities, while, between anchorages, the strand was unbonded to assure uniform stresses along its length. Epoxy resin and injection equipment's were supplied by HILTI, including adhesive dispenser, mixing nozzle, injection tubes, cleaning brush, etc.



a) b) c) d)

Figure 3 Specimen CB2: a) HILTI diamond drilling core bit, b) HILTI drilling machine, c) cross section of the strand and drilled hole, d) application of the prestressing force

a) b)

Figure 4 Specimen CB2: strands cut at the edges of the specimen at: a) column and b) beam face

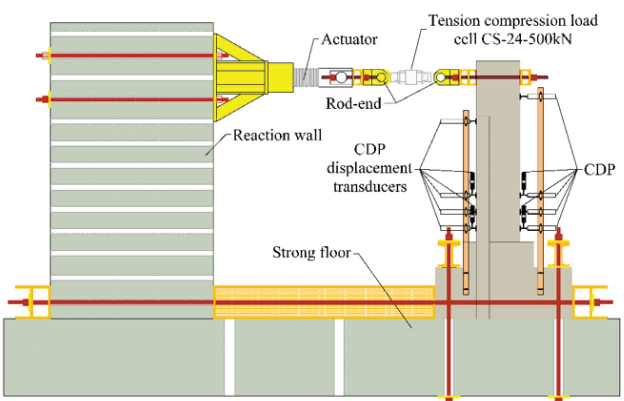
The necessary curing time of resin is around 24 hours, but the temporary anchorage system shown in Figure 3d was removed after 48 hours. The prestressing force was transmitted from prestressing steel strands to concrete through bonding along the anchorages' lengths. Finally, the excessive strands length, which remained out of the beam cross section, was cut as shown in Figure 4a, b and the cross-section geometry of the strengthened specimen remained unchanged.

This strengthening method presents almost all the advantages that the conventional method of strengthening by prestressing has, but it does not have the disadvantage of visible anchorages. Furthermore, this strengthening method does not increase the permanent loads, which is one of the main disadvantages of some other strengthening methods. This method is aesthetic, because it does not impose interventions in the existing interior architectural design. The intervention may be local (applied only in the deficient beams) or in the entire building.

2.2 Test setup

The experimental campaign was carried out in the Laboratory of Heavy Structures of the Faculty of Science and Technology – Universidade NOVA de Lisboa. The laboratory has two reaction walls and a strong floor. Specimens were fixed vertically in the strong floor and horizontally to the reaction wall (both by post tensioning bars), as shown in Figure 5a. Horizontal load was applied by an actuator with ± 500 kN load capacity and 500 mm (± 250 mm) displacement capacity.

Specimens instrumentation is shown in Figure 5a, 16 strain gauges were installed in specimen CB1 to monitor the reinforcement strain, and 26 in specimen CB2. In both specimens, 11 CDP TML strain guage transducers were installed. Figure 5b shows a general view of specimen CB1 after instruments installation. Specimen CB2 had similar instrumentation.



a) b)

Figure 5 a) Test setup for both specimens; b) instrumentation of specimen CB1

2.3 Loading history

The load test procedure (Gião *et al.* [9]) is shown in Figure 6. The first step corresponds to the imposition of a pre-established gravity load, followed by the subsequent steps (where FC means force-controlled, DC means displacement-controlled):

i) (DC) Imposition of a pre-established displacement (+ Δ);
ii) (FC) Unloading until the value of the gravity load is re-established;
iii) (DC) Imposition of a pre-established displacement-controlled unloading (-Δ);
iv) (FC) Loading until the value of the gravity load is re-established.

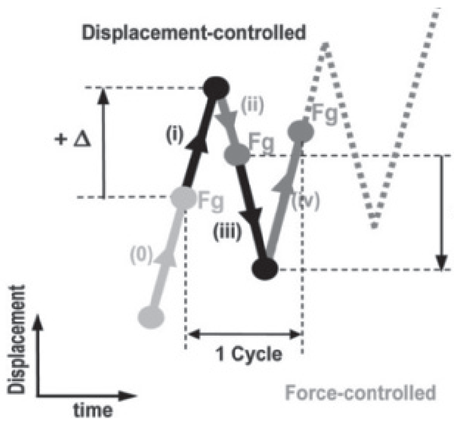
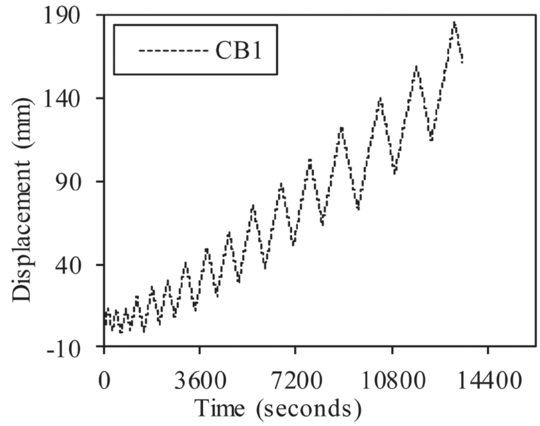


Figure 6 Typical load cycle of experimental loading history [9]

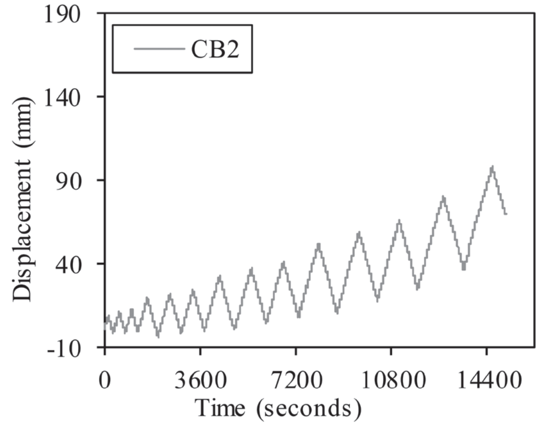
The loading history used for both specimens consisted in the imposition of the described procedure. Each amplitude displacements are repeated for three cycles, starting from the reference amplitude: $\Delta = \pm 1.0 \cdot d_0, \pm 2.0 \cdot d_0, \pm 3.0 \cdot d_0, \pm 4.0 \cdot d_0, \pm 5.0 \cdot d_0, \pm 6.0 \cdot d_0, \ldots$, up to failure. Evaluation of the cycle's amplitude was based on first yield displacement, which was estimated experimentally before starting the cyclic test of the reference beam CB1. First yield displacement was estimated by monitoring the strain in the longitudinal reinforcement while imposing horizontal displacement to the specimen [9].

Specimen CB1 displacement at yielding was d_y = 14 mm for negative bending moments. The reference displacement amplitude was selected half of the first yield of negative moments (d_{0CB1} = 7 mm). The same amplitude was maintained for the strengthened specimen CB2.

The loading histories of specimens CB1 and CB2 are given in Figure 7a and Figure 7b, respectively. The imposed gravity load corresponds to 50 % of the yielding flexural moment calculated analytically (strain hardening was not taken into account). The imposed gravity load was 40 kN for both specimens, as shown in Figure 9.



a)



b)

Figure 7 Loading history of the cyclic test including gravity load effects: a) specimen CB1; b) specimen CB2

3 Experimental results

3.1 Specimen CB1

Specimen CB1 (reference specimen) failed in tension in the 16th cycle by rupture of four slab longitudinal steel reinforcing bars, Figure 8d, situated only on one side of the flange due to non-symmetrical behaviour. The load-displacement diagram is presented in Figure 8a and its failure mode has been described in detail in Muhaj *et al.* [15]. After failure, high residual deformations were observed due to significant crack openings in the tensile region, Figure 8b, c. Light damage was observed in the compressed region, situated mainly in the cover region outside of the confined concrete and no buckling of reinforcement in compression was observed, shown in Figure 8b, e, f.

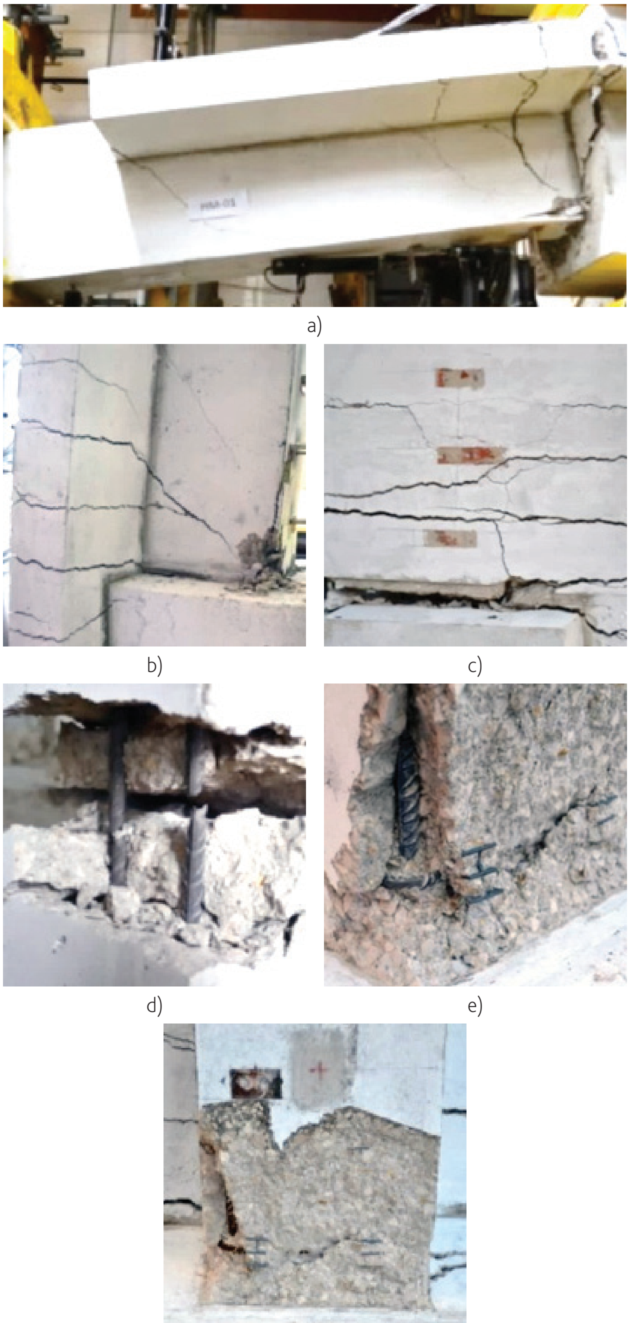


Figure 8 Reference specimen CB1 after failure

3.2 Specimen CB2

In specimen CB2 a different failure mode from specimen CB1 was observed, due to the induced pressing force applied for strengthening. Specimen CB2 failed in compression by buckling of compressed reinforcement and crushing of concrete, as shown in Figure 10b, e, f. After attaining the maximum load for the intended 3.5 % beam rotation, specimen strength and stiffness degradation was noticed.

Bonded anchorages slip became significant when load bearing capacity was reached. In this case, slip of steel strands led to a prestressing force lost and all the effectiveness of the strengthening solution. This phenomenon was observed in the 13th cycle when the steel strands had significant slip. The compressed region deteriorated, and the load decreased beyond 85 % of its maximum value. Then, rupture of tensile slab reinforcing bars was observed, Figure 10d.

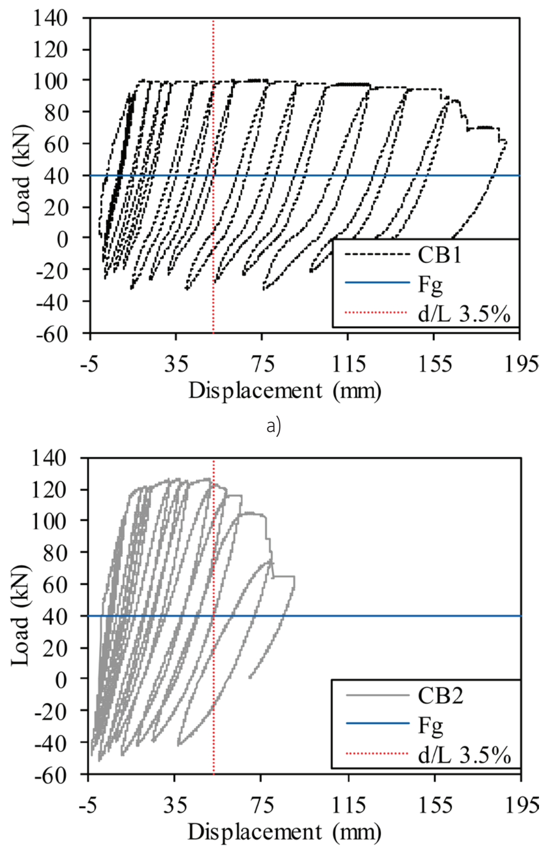


Figure 9 Load – Displacement diagrams: a) reference specimen CB1, and b) strengthened specimen CB2

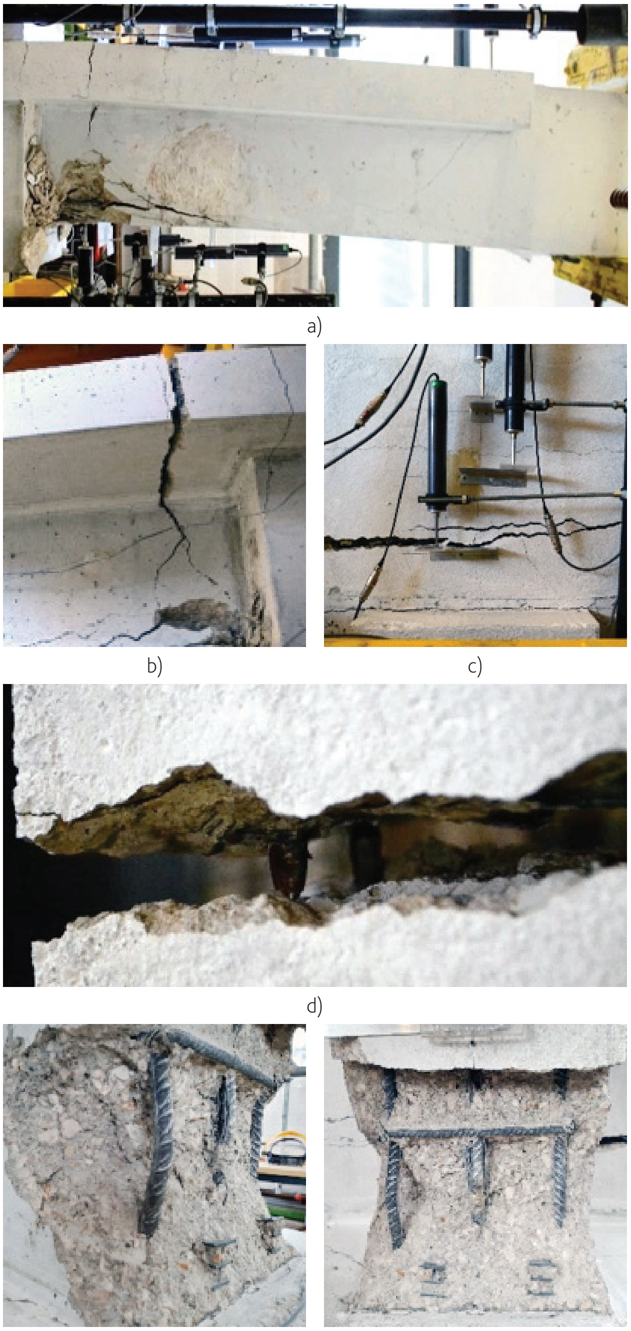


Figure 10 Strengthened specimen CB2 after failure

3.3 Energy dissipation

Energy dissipation per cycle and per half cycle up to failure for both specimens have been plotted in diagrams of Figure 11. The diagrams plotted in Figure 11a include all the cycles of the experimental campaign, till the test was stopped; whereas in Figure 11b there are shown only the cycles prior to failure. Specimen CB2 dissipated more energy per each cycle than specimen CB1, Figure 11a. For the same intended beam rotation of 3.5 %, Figure 11b, the strengthened specimen CB2 accumulated 225 % more energy than the reference specimen CB1. Furthermore, CB2 was also subjected to more cycles than the reference specimen up to the beam rotation of 3.5 % (12 cycles and 10 cycles). The results are summarized below in Table 2.

Table 2 Comparison between reference specimen CB1 and strengthened specimen CB2

Specimen Identific.	F_{max} (kN)	F_{max} ratio	Up to *d/L* 3.5%			Up to failure					Number of cycles up to failure
			d/L (%)	Diss. Energy (kNm)	Energy ratio	*d/L* (%)	Displ. (mm)	*d/L* ratio	Diss. Energy (kNm)	Energy ratio	
CB1	100.3	1.00	3.50	4.23	1.00	8.45	126.7	1.00	11.86	1.00	15
CB2	126.9	1.27	3.52	9.50	**2.25**	3.00	45.0	**0.35**	10.06	**0.85**	12

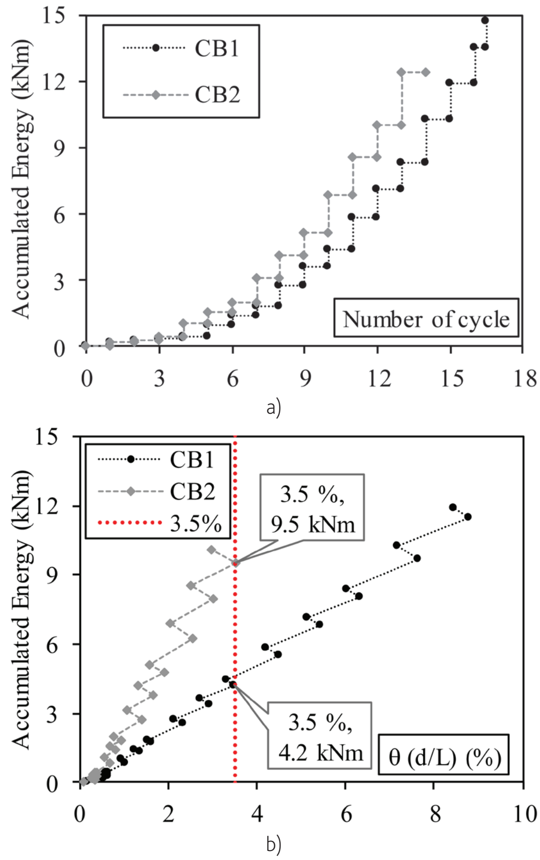


Figure 11 Specimens CB1 and CB2: Accumulated energy – a) number of cycles, b) "drift d/L"

The application of the proposed strengthening method in a full frame would be very interesting for investigation, but due to laboratory restrictions and funding, it is out of the scope of this project. Nevertheless, this is the subject of an ongoing numerical investigation and similar results are predicted to be observed. Further experimental and numerical investigation is required before reaching a firm conclusion.

4 Conclusions

The proposed strengthening technique improved significantly the seismic behaviour of RC beam, by enhancing load capacity and decreasing the residual deformations. Up to "drift d/L" 3.5 %, the dissipated energy of specimen CB2 was 225 % of specimen CB1.

The strengthening technique may be adopted for different beam cross-sections, span lengths, and reinforcing details. The strengthened specimen behaviour might be improved by increasing the anchorage length (increases load capacity), strand placement, prestressing force or prestressing strand characteristic to follow the design requirements.

Application of the proposed strengthening method (internally installed prestressing strands) does not increase permanent loads on the existing structures as it does not require increase of the existing beam cross-section. Furthermore, this strengthening technique does not impose interventions in the existing interior architectural design and it is aesthetic.

The intervention may be local, applied only in the deficient beams, or in the entire building. Detailed site survey should be carried out for defining the hole geometry, in order not to interfere with the reinforcement of the beam during drilling. This application is fast and easy if compared to other existing strengthening methods.

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