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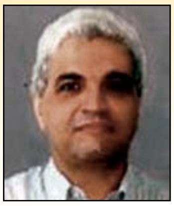
Experimental Analysis of Reinforced Concrete Columns Strengthened with Self-Compacting Concrete

Análise Experimental de Pilares de Concreto Armado Reforçados usando Concreto Auto-Adensável


M. Y. M. OMAR [a]
mohaomar@gmail.com

R. B. GOMES [b]
rbggomes@gmail.com

A. P. A. REIS [c]
andrea.liserre@gmail.com

Abstract

This paper presents the results of reinforced concrete columns strengthened by addition of a self-compacting concrete overlay at the compressed and at the tensioned face of the member, with and without addition of longitudinal steel bars. Eight columns were submitted to loading with an initial eccentricity of 60 mm. These columns had 120 mm x 250 mm of rectangular cross section, 2000 mm in length and four longitudinal reinforcement steel bars with 10 mm in diameter. Reference columns P1 and P2 were tested to failure without any type of rehabilitation. Columns P3 to P8 were loaded to a predefined load (close to the initial yield point of tension reinforcement), then unloaded and strengthened for a subsequent test until failure. Results showed that the method of rehabilitation used was effective, increasing the loading capacity of the strengthened pieces by 2 to 5 times the ultimate load of the reference column.

Keywords: column; reinforced concrete; strengthening; eccentricity; self-compacting concrete.

Resumo

Este artigo apresenta os resultados de pilares de concreto armado reforçados por meio da adição de camadas de concreto auto-adensável, tanto na face tracionada quanto na face comprimida, com ou sem adição de barras de aço longitudinais. Oito pilares foram submetidos a um carregamento com excentricidade inicial de 60mm. Estes pilares possuíam seção transversal retangular de 120 mm x 250 mm, comprimento de 2000 mm, e armadura longitudinal constituída por quatro barras de 10 mm de diâmetro. Os pilares de referência, P1 e P2, foram ensaiados até a ruptura sem nenhum tipo de reabilitação. Os pilares P3 à P8 foram inicialmente submetidos a um pré-carregamento que provocava na armadura longitudinal, tensões próximas a do escoamento do aço. Em seguida, estas peças foram descarregadas e reforçadas. Após o concreto do reforço atingir resistência adequada, estes pilares foram ensaiados até a ruptura. Os resultados mostraram que o método de reforço estudado foi eficiente, pois dependendo da face do pilar em que se aplicava a camada de reforço, as peças reabilitadas apresentaram uma capacidade de carga de 2 a 5 vezes maior que a dos pilares não reforçados.

Palavras-chave: pilar; concreto armado; reforço; excentricidade; concreto auto-adensável.

[a] *School of Civil Engineering, Federal University of Goias, mohaomar@gmail.com, Goiânia, Brazil.*
[b] *School of Civil Engineering, Federal University of Goias, rbggomes@gmail.com, Praça Universitária, s/nº – Setor Universitário – CEP 74605-220, Goiânia, Brazil;*
[c] *School of Civil Engineering, Federal University of Goias, andrea.liserre@gmail.com, Praça Universitária, s/nº – Setor Universitário – CEP 74605-220, Goiânia, Brazil;*

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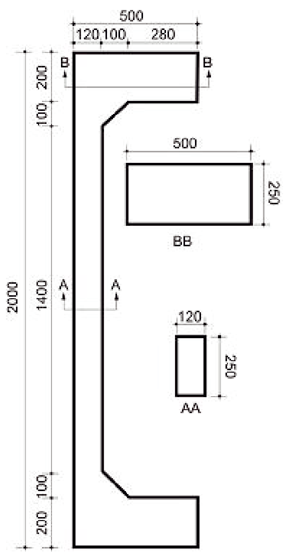
1. Introduction

The repair, rehabilitation and upgrading of reinforced concrete structures is a major challenge to civil engineers. Thus, the knowledge of upgrading or strengthening techniques of reinforced concrete structures is very interesting for engineering purposes. Despite the considerable development in this area in the last years, many professionals still use methods based only on empirical experience. Probably this happens because each structure has individual characteristics which demand a specific rehabilitation process. Although numerous structures have been rehabilitated adding new material to the concrete section, limited data are available on their performance under applied loads until failure.
Aiming to increase the knowledge about the structural behavior of rehabilitated constructions, this paper analyzes the behavior of columns strengthened by an increase of the cross section with or without the addition of longitudinal steel bars embedded by self-compacting concrete (SCC). The re-casting of the cross section does not necessarily have to be applied all around the column, thus it was studied the rehabilitation adding new concrete restricted to some of its faces.
The guarantee of a satisfactory bond and a monolithic strengthening depends on the correct treatment of the old concrete surface and substrate cleaning and saturation before the subsequent placing of the new concrete layer and steel bars. Besides that, differences in properties of the new concrete and original concrete must be considered, especially concerning elastic modulus, shrinkage and creep strains. To match properties of the new repair material as closely as possible to the original concrete of the structure, the use of Portland cement concrete or similar cementitious compositions are frequently the best choices (Clímaco [1]).

Figure 1 – Dimensions of the original columns (mm)



In order to obtain a material with mechanical properties similar to original concrete, to facilitate the application of the strengthening overlay and to avoid execution defects, self-compacting concrete (SCC) may be employed as a strengthening material. This type of concrete may be cast on formwork, thus filling all empty spaces without the aid of external compaction or vibration. It provides fluidity, cohesion, and segregation resistance.
Silica fume is used as an admixture to concrete to improve the engineering properties. According to Aïtcin [2], the silica fume added to the concrete improves the paste-aggregate interface zone, adherence, impermeability, axial compression strength, and cohesion of fresh concrete, avoiding exudation. Thus, additions of silica fume may improve the bond conditions between the new concrete and the substrate. It is desired that strengthened elements acts like monolithic structures guarantying the complete load transfer from the base concrete to the new material in the vertical joint region, to avoid the debonding between these materials.
The shear resistance of the joint is the sum of the contribution of the cohesion between the two concretes of the joint, the concrete-to-concrete friction resistance mobilized when the joint is simultaneously subject to shear and normal compression, and the dowel action of the reinforcing bars crossing the interface.
If the adherence between contact surfaces fails to ensure the monolithicity of the rehabilitation, it is possible to increase the joint shear strength using steel connectors crossing the interface. Such transverse reinforcement is normally used in composite precast parts, in which tangential stresses show high values because of a small contact surface between concretes cast at different ages (El Debs [3]).
Abu-Tair et al. [4] evaluated the surface roughness of concrete of three different repair materials: epoxy resin, a modified cementitious material, and Portland cement concrete. It also was analyzed four types of substrate surface preparation: manual removal of the superficial concrete layer with hammer and chisel, electric hammering, wire brushing and no intervention. The most successful results of adherence strength were obtained by the Portland cement concrete repair, with a surface treated with hammer and chisel and no adhesive agent.
Adorno [5] and Araújo [6] have carried out tests with reinforced concrete columns submitted to combined bending and axial compression. Their experiments have shown the influence of load eccentricity variation on a column's load-bearing capacity. The selection of a 60 mm load eccentricity in this study was supported by the results of such experiments. This eccentricity allowed the tested columns' reinforcements to reach their yield.
This paper highlights the study of reinforced concrete columns strengthened with self-compacting concrete (SCC) at the compressed face or at the tensioned face and at the compressed and tensioned faces simultaneously; with all columns submitted to combined bending and axial compression at an initial eccentricity of 60 mm. Depending on the strengthened face this eccentricity may increase or decrease in the post-strengthening test. Our specific aims were: 1) to analyze the effect of load (pre-loading) on the column prior to strengthening; 2) to check experimentally whether strengthened columns may be regarded as monolithically cast columns, in view of a perfect adherence between strengthening material and the substrate; 3) to verify the influence of strengthening on columns as regards faces (tension or compression), thickness, and consequent eccentricity variation.

2. Materials and experimental program

Eight columns (named Original Columns) of 120 mm x 250 mm rectangular cross section, and 2000 mm in length were tested (Omar [7]). The concrete mix design aimed a 28 days compressive strength of 30 MPa and an elastic modulus of 28 GPa. During each concrete casting cylindrical specimens (150 mm x 300 mm) were moulded to determine the compressive strength of the concrete.

The longitudinal reinforcement of the original columns was composed of four 10 mm diameter bars. These elements were designed to be submitted to eccentric compression (initial eccentricity of 60 mm). Full details of columns dimensions and bar reinforcement are shown in Figures [1] and [2], respectively.

Two testing stages were performed. In the first stage, two reference columns were tested to failure (P1 and P2) and the remaining six (P3 to P8) were submitted to loading which corresponded to a strain close to the initial yield point of the tension reinforcement. The second stage consisted of testing columns to failure after strengthening.

Strengthening was performed with unloaded columns positioned outside the test apparatus. Columns P3 and P4 were strengthened on the tension face adding longitudinal steel bars measuring 2 φ 10.0 mm and 2 φ 12.5 mm. These new bars were encased by a 45

Figure 2 – Detailing of the reinforcements of the original columns (mm)

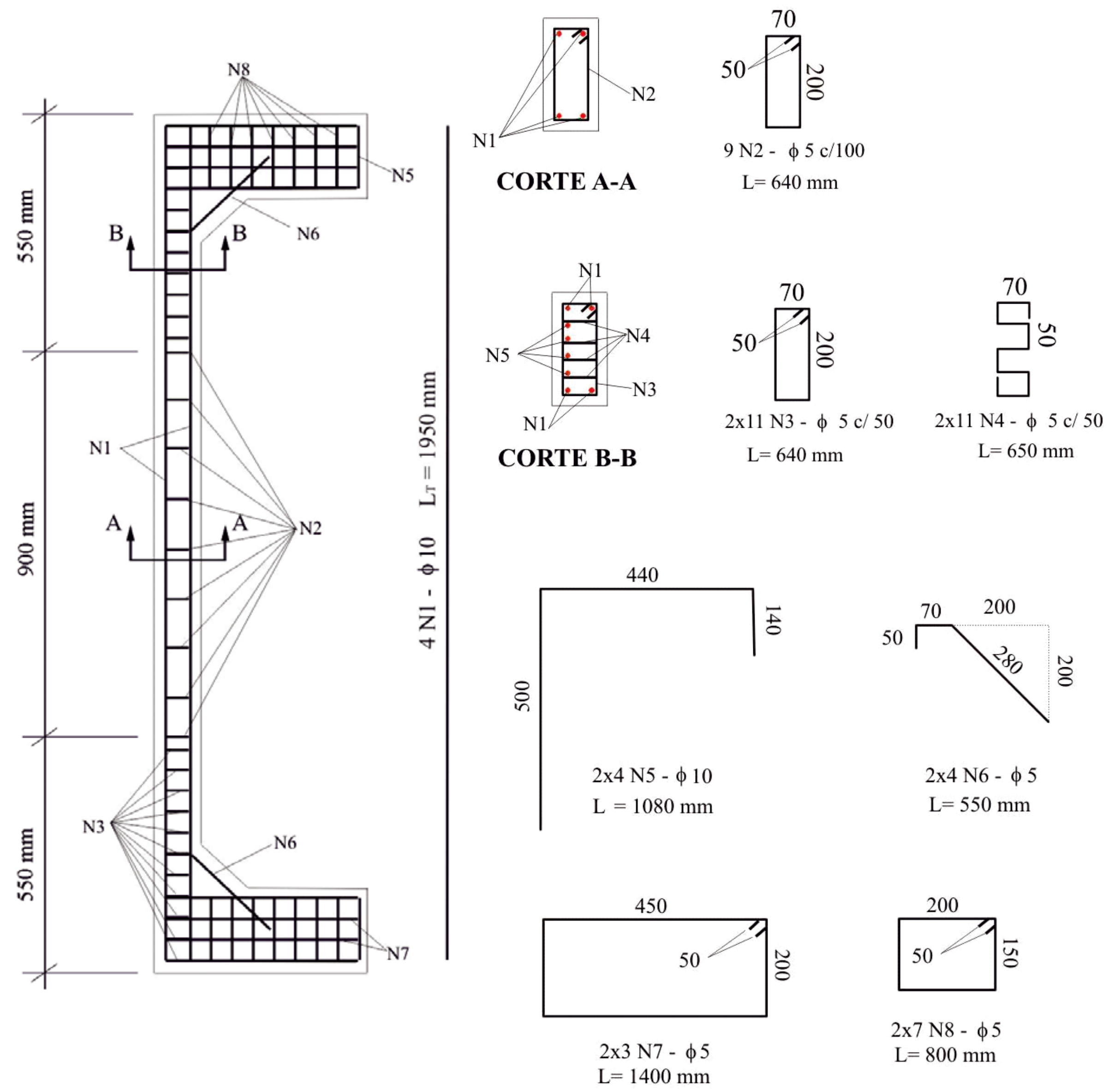


Table 1 – Main characteristics of tested columns

Type	1st Phase $e_{inicial}$= 60,0 mm	2nd Phase					
	Name	Name	$e_{r,in}$ (mm)	E_{rc} (mm)	R_t	E_{rt} (mm)	h (mm)
Reference (1st phase)	P1	-	-	-	-	-	120
	P2						
Original (1st phase) Strengthened (2nd phase)	P3	PT10	82.5	-	2ϕ10.0mm	45	165
	P4	PT12			2ϕ12.5mm		
	P5	PC45T10	60.0	45	2ϕ10.0mm	45	210
	P6	PC45T12			2ϕ12.5mm		
	P7	PC35	42.5	35	-	-	155
	P8	PC55	32.5	55			175

$e_{inicial}$ = Initial eccentricity (P = 0 kN);
$e_{r,in}$= Initial eccentricity of strengthened columns (P = 0 kN);
E_{rc} = Thickness of strengthening on the compressed face;
R_t = Strengthening reinforcement on the tension face;
E_{rt} = Thickness of strengthening on the tension face;
h = Total height of the cross section of the column.

mm thick SCC layer. These columns were thus called PT10 and PT12, respectively. Their initial eccentricity increased to 82.5 mm due to strengthening.
Columns P5 and P6 were strengthened on the tension faces adding longitudinal bars measuring 2 φ 10.0 mm and 2 φ 12.5 mm respectively, and with 45 mm thick SCC layers on both compressed and tension faces. They were respectively named PC45T10 and PC45T12, and their initial eccentricity of 60 mm was maintained.
The values of yield strength of the steel bars were 613 MPa and 619 MPa for 10 mm and 12.5 mm diameter bars, respectively. The strain corresponding to yield strength was 2.87 mm/m for 10 mm diameter bar and 2.53 mm/m for 12.5mm diameter bar.
Columns P7 and P8 were strengthened on the compressed faces with 35 mm (PC35) and 55 mm (PC55) thick SCC layers without using any steel bars. Consequently their initial eccentricities were reduced to 42.5 and 32.5 mm, respectively.
Table [1] presents the main characteristics of the tested columns and Figure [3] reveals the detail of the cross sections of original and strengthened columns. Table [2] shows the proportion of materials per cubic meter of concrete used in the substrate and the strengthening.
The entire substrate surface submitted to strengthening was manually treated with hammer and chisel to expose the aggregate for proper adhesion of the rehabilitation concrete. The substrate surface was cleaned and saturated prior to strengthening. Steel U-bolts of 5 mm in diameter, spaced out at 50 mm intervals, and glued with epoxy resin were placed within holes made by 8 mm thick and 50 mm deep drill bits.

The original columns were submitted to eccentric compression at an initial eccentricity of 60 mm. The initial eccentricity varied among the strengthened columns in terms of thickness and face strengthened. The load was applied by a hydraulic actuator which was activated by a hydraulic hand pump. Load cells of 500 kN were placed on the upper and lower ends of the columns whereas a single 1000 kN load cell was placed on the upper end of columns PC45T10, PC45T12, and PC55. A frame anchored to the strong floor was used for the reaction of specimens.
A digital dial gauge of 0.01 mm precision (R1) measured the horizontal displacement at the middle section of specimens. Figure [4] shows the test scheme with the positions of load cells and of the comparing watch, as well as a photograph with details of the equipment used for the measurements, load application, and bonding of an original column within the testing system.
Electric strain gauges connected to the data acquisition system, were used to monitor steel and concrete strains on the middle section of specimens. Figure [5] presents the position of the extensometers on the steel and concrete of all the columns. Throughout the analyses, reinforcement strains on tension (T) and compressed (C) faces were the averages of readings supplied by the extensometers.
In all columns, the concrete used had cylinder compressive strength of the substrate between 25.1 (PC35 and PC55) MPa and 30.8 MPa (PC45T12) and the compression strength of the strengthening concrete varied from 43.0 MPa to 46.8 MPa (Table 3).

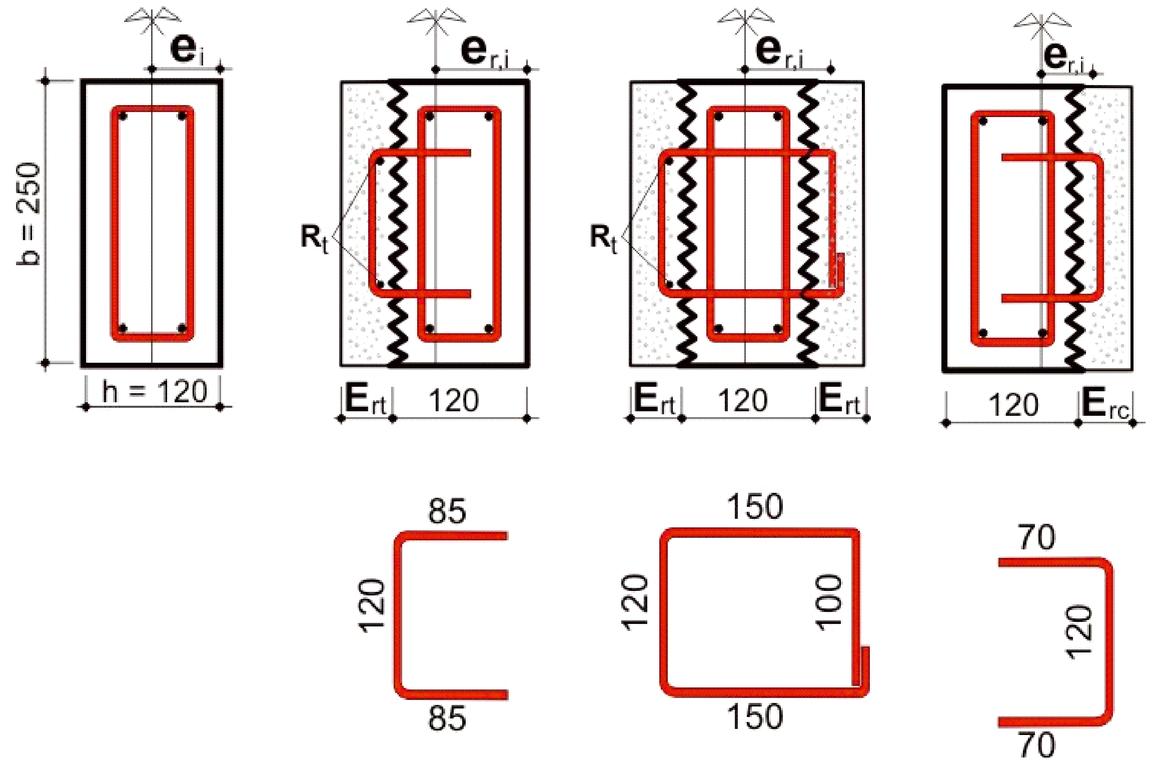
3. Results and discussions

3.1 Ultimate load and mode of failure

Columns strengthened on the tension face, PT10 and PT12, had ultimate loads of 268 kN and 280 kN respectively, that are 2.1 and 2.2 times greater than the failure load of the reference column (P1). This load gain was a result of the increase of the cross section and of the reinforcement rate, despite the increase of the initial eccentricity of the force applied. The failure load of column PT12 was 12 kN higher than that of column PT10, even though a higher longitudinal reinforcement rate was added to it. The failure of PT10 and PT12 were ductile and occurred by yielding of the longitudinal steel bars at the tension face, followed by the crushing of compressed

Figure 3 – Cross section of tested specimens

Reference and original column (P1, P2, P3, P4, P5, P6, P7 and P8)	Column strengthened by tension (PT10 and PT12)	Column strengthened by tension and compression (PC45T10 and PC45T12)	Column strengthened by compression (PC35 and PC55)



	Connector – Column strengthened by tension	Connector – Column strengthened by tension and compression	Connector – Column strengthened by compression
	ϕ 5.0 mm c. 50mm, L = 290 mm	ϕ 5.0 mm c. 50mm, L = 570 mm	ϕ 5.0 mm c. 50mm, L = 260 mm

e_i = Initial eccentricity;
$e_{r,i}$ = Initial eccentricity of strengthened columns;
E_{rc} = Thickness of strengthening on the compressed face;
E_{rt} = Thickness of strengthening on the tension face;

Table 2 – Proportion of substrate and strengthening materials per m^3 of concrete

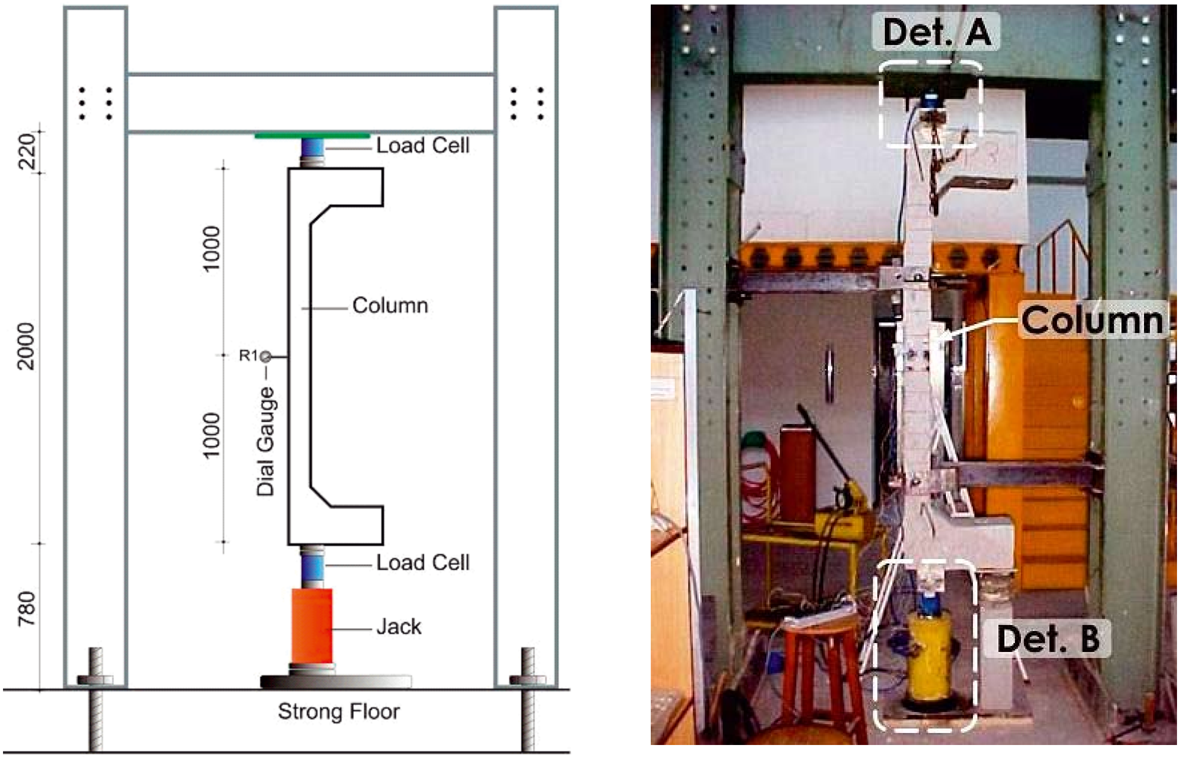
Materials	Substrate Quantity (p/m^3)	Strengthening (SCC) Quantity (p/m^3)
Cement	310 kg	539 kg
Natural Fine Sand	155 kg	0 kg
Natural Coarse Sand	233 kg	191 kg
Artificial Sand	497 kg	-
Filler	-	355 kg
Aggregates ($\phi \leq$ 12.5mm)	388 kg	1043 kg
Aggregates ($\phi \leq$ 19 mm)	619 kg	0 kg
Water	155 ℓ	223 ℓ
Superplasticizer	-	3.77 kg (1.4% on cement)
Retarder Additive	2.17 ℓ	1.08 kg (0.7% on cement)
	Slump (95 ± 10) mm	Slump flow test = 75 cm

concrete in the mid-span region, at the compressed face. Table [3] shows ultimate loads and modes of failure of all tested columns.
Columns PC45T10 and PC45T12 revealed ultimate loads almost 5 times greater than that of the reference column P1 (645 kN and 630 kN, respectively). This load gain was a result of the increase of the cross section and of the reinforcement rate. The failure was sudden by the crushing of compressed concrete at the mid-span region of the column without the yield of the tension reinforcement. There were no traces of debonding of the strengthening concrete.
Columns PC35 and PC55 – strengthened on the compressed face with SCC layers of 35 mm and 55 mm, respectively – showed ultimate loads 2.9 and 3.9 times greater than that of the reference column (P1), respectively. This load gain occurred due to the increase of the cross section and to the reduction of the initial eccentricity of the force applied.
Columns PC35 and PC55 failed prematurely due to the debonding of the strengthening concrete and lack of yield of the tension reinforcement. The debonding of the strengthening concrete of column PC35 occurred close to the lower end of the column, whereas the debonding of the strengthening concrete of column PC55 occurred close to its upper end. The load difference between columns PC35 and PC55 was 126 kN, representing 33% more for the latter. This highlights the influence of initial load eccentricity on the columns and the increase in cross section. Figure [6] presents photographs of columns PC35 and PC55 after failure.
In addition, even though columns PT10 and PT12 were strengthened with greater thickness than column PC35, the failure load was lower because the strengthening was placed on the tension face. This reveals that the reduction of initial eccentricity may benefit the load-bearing capacity of the strengthened specimens.
Columns PC35 and PC55, which were strengthened on the compressed face even with the debonding of the strengthening concrete, recorded greater load gain than the columns strengthened on the tension face. Such an increase was a result of the increase in cross section and of the reduction of the initial load eccentricity. If the debonding of the strengthening concrete had not occurred, columns PC35 and PC55 might have revealed higher failure loads.

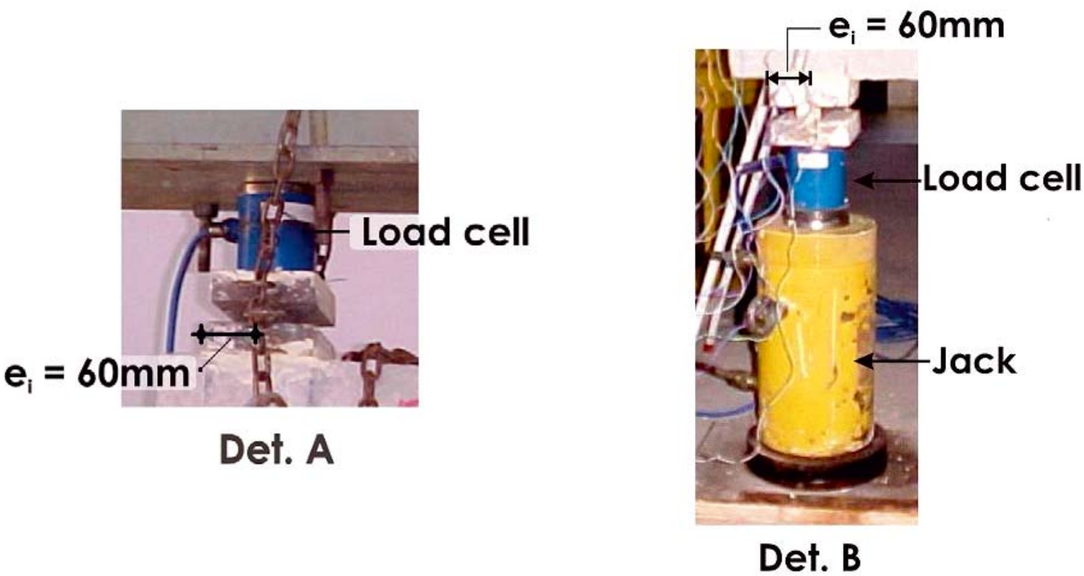
3.2 *Horizontal displacement of the column*

The reference columns (P1 and P2) registered maximum horizontal displacements at mid-heights of 20.35 mm and 37.63 mm, respectively. The columns strengthened on the tension face, PT10 and PT12, had maximum displacements of 32.58 mm and 20.37 mm, respectively, whereas columns PC35 and PC55 – strengthened on the compressed face – reached maximum displacements of 22.05 mm and 13.62 mm, respectively. Column PC45T12 had the lowest horizontal displacement among all the columns (11.67 mm). Column PC45T10 reached a horizontal displacement of 19.46 mm.
Figure [7] shows the load-horizontal displacement curves for all the columns. Columns PT10 and PT12 had similar stiffnesses which were higher than that of the reference columns. This probably occurred because they had larger cross sections and higher reinforcement rates. However, these specimens had lower stiffness values than all the other columns strengthened on the compressed face. As regards the failure load of column P1, columns PC45T10, PC45T12, PC35, and PC55 had similar stiffnesses with displacements of approximately 14.0 times lower than that of column P1.
Column PC35, which had a lower cross section than that of columns PT10 and PT12, had higher stiffness. The initial eccentricity

Figure 4 – Scheme and photograph of the column testing system (mm)



(a) Scheme of testing



(b) Details

of column PC35, lower by 17.5 mm, generates a lower bending moment and consequently smaller displacements. The horizontal displacement measured at the failure of column PC35 was approximately 5 times greater than the displacements measured on columns PC45T10 and PC45T12.

3.3 Steel strain – Tension face reinforcement

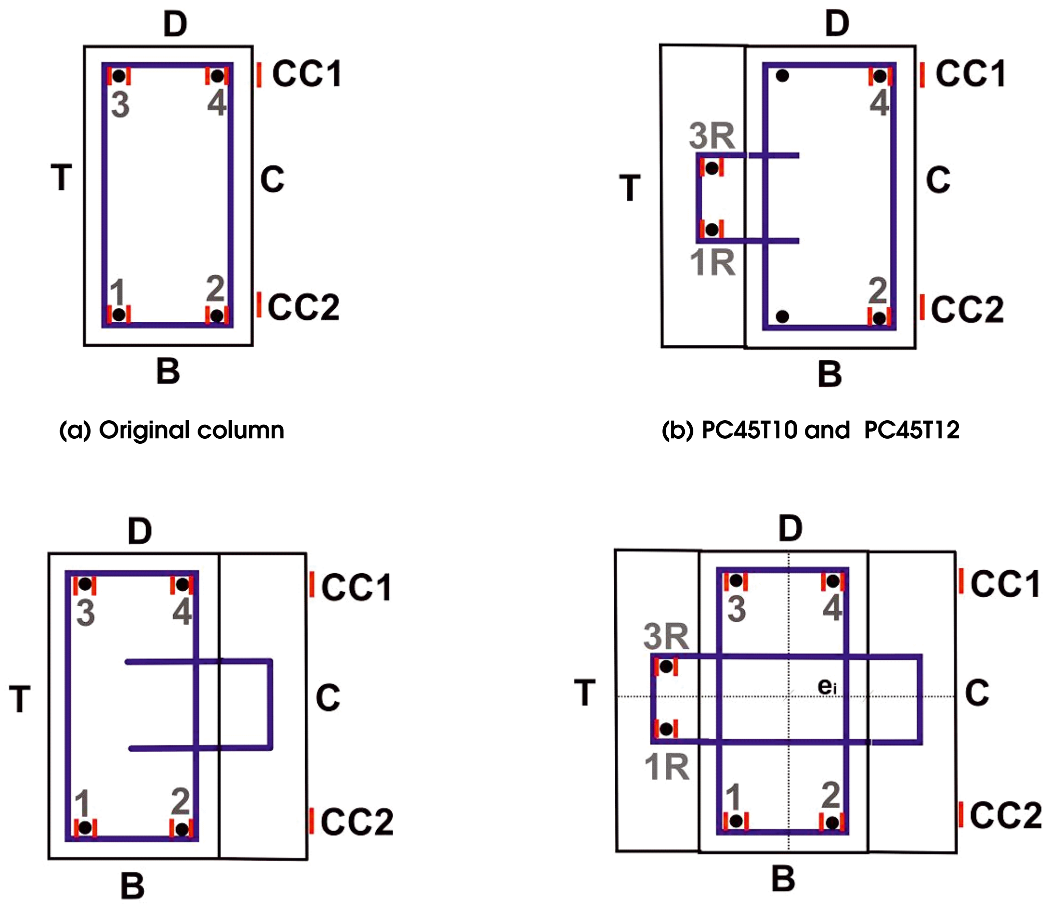
The reference columns (P1 and P2) showed tension steel yield and registered maximum strains of 2.89 mm/m and 3.71 mm/m, respectively. The columns strengthened on the tension face, PT10 and PT12, had maximum strains of 3.35 mm/m and 2.63 mm/m respectively, thus indicating the yield of the tension steel. The graph in Figure [8] shows the load-strain curve of the tension steel of all the columns tested.

Maximum strains of 1.47 mm/m and 1.20 mm/m were respectively registered for columns PC35 and PC55 (strengthened on the compressed face), whereas maximum strains of 1.70 mm/m and 1.61 mm/m were respectively registered for columns PC45T10 and PC45T12 (strengthened on both compressed and tension faces). These values indicate that the tension steel of all columns had not yet reached the yield point before failure.

Figure [8] shows that column PC35 – with a smaller cross section – had smaller steel strains than columns PT10 and PT12. As regards the failure load of column PC35, columns PC45T10 and PC45T12 had smaller strains than one third of the strains of column PC35. This was probably due to the fact that columns PC45T10 and PC45T12 had a larger cross section and a higher reinforcement rate than PC35.

The tension reinforcement strain of column PT10 was approximately 30% higher before failure than the maximum strain of column PT12. The strains of columns PC45T10 and PC45T12 (strengthened on compressed and tension faces) were similar, despite a 2.3% difference in failure load and a higher reinforcement rate (1.56 times higher) of PC45T12. Even though columns PC35 and PC55 (strengthened by compression) had lower failure loads than columns PC45T10 and PC45T12 (strengthened by compression and tension), the tension strains measured were lower for PC35 up

Figure 5 – Positions of strain gauges (T Face: tension face; C Face: compressed face)



(a) Original column

(b) PC45T10 and PC45T12

(c) PC35 and PC55

(d) PC45T10 and PC45T12

Table 3 – Failure loads and modes of reference and strengthened columns

Column	f_c^{sub} (MPa)	f_c^{refor} (MPa)	P_u (kN)	$e_{inicial}$ (mm)	D_{max} R3 (mm)	e_{final} (mm)	$\varepsilon_s max$ (mm/m)	$\varepsilon_c max$ (mm/m)	Failure mode
P1	30.5	-	130.1	60.0	20.35	80.35	2.89	-2.22	YC
P2	30.6	-	157.0	60.0	37.63	97.63	4.36	-4.69	YC
PT10	30.7	43.0	268.0	82.5	32.58	115.08	3.36	-3.45	YC
PT12	30.7	44.1	280.0	82.5	20.37	102.87	2.90	-2.92	YC
PC45T10	30.7	44.8	645.0	60.0	19.46	79.46	1.75	-3.57	C
PC45T12	30.8	45.5	630.0	60.0	11.67	71.67	1.72	-3.89	C
PC35	25.1	46.2	380.0	42.5	22.05	64.55	1.47	-2.43	De
PC55	25.1	46.8	506.0	32.5	13.62	46.12	1.43	-2.49	De

f_c^{sub}: Compression strength of the substrate on the day of failure testing (for reference columns) and of pre-damage (for strengthened columns);
f_c^{refor}: Compression strength of the strengthening concrete on the day of testing;
P_u: Failure load;
$e_{inicial}$: Initial eccentricity; e_{final}: Final eccentricity = $e_{inicial}$ + R3;
R3: Maximum horizontal displacement;
$\varepsilon_s max$: Maximum strain of tension steel; $\varepsilon_c max$: Maximum strain of compressed concrete;
M_u: Maximum moment = $P_u \cdot e_{final}$;
YC: Steel yield and concrete crushing; C: Concrete crushing; De: debonding of strengthening;
ε_y for ϕ10.0 mm = 2.87 mm/m; ε_y for ϕ 12.5 mm = 2.53 mm/m;

to the 340 kN load and for PC55 up to the 460 kN load. This was probably a result of lower initial eccentricities for columns PC35 and PC55 (42.5 mm and 32.5 mm, respectively).

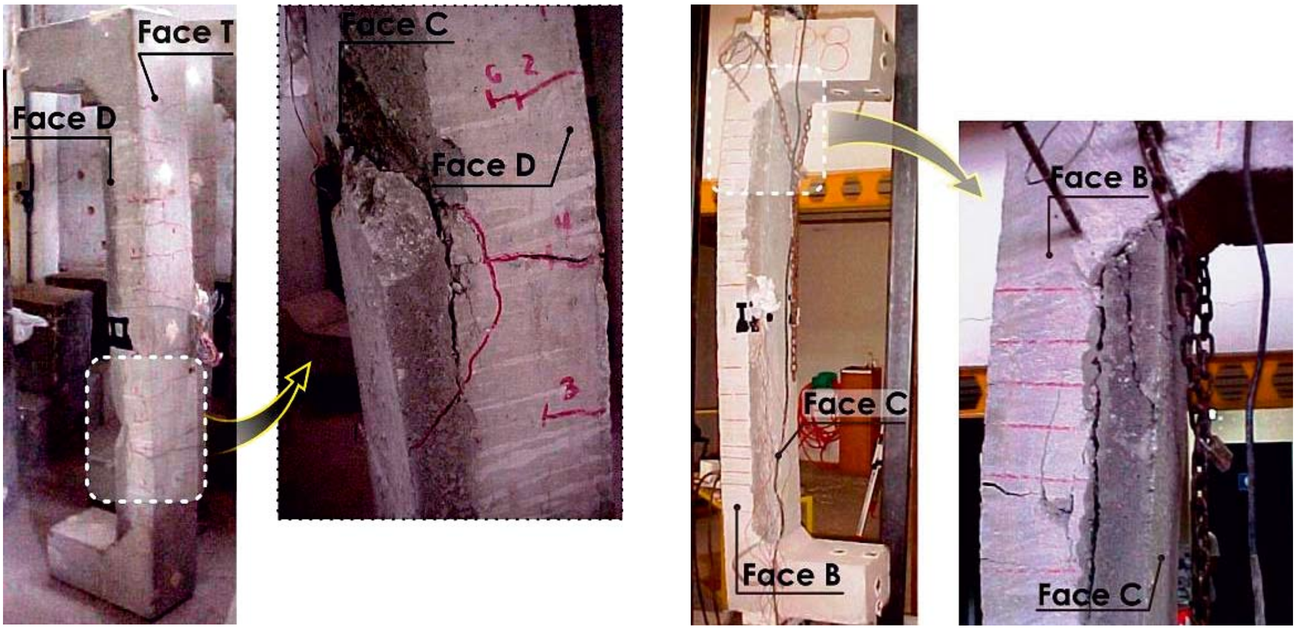
3.4 Steel strain – Compressed face reinforcement

The reference columns (P1 and P2) and columns PC35 and PC55 (strengthened on the compressed face) had maximum compressed steel strains lower than 1.0 mm/m. Columns PC45T10 and PC45T12, strengthened on both compressed and tension faces, had maximum strains of 1.78 mm/m and 1.71 mm/m respectively, whereas columns PT10 and PT12, strengthened on the tension face, reached maximum strains of up to 3.89 mm/m.
The graph in Figure [9] shows the load-strain curve of the compressed steel of all the columns tested. As regards columns P1, PT10, and PT12, the compressed reinforcement revealed, since the start of testing, higher strains than those observed in columns PC45T10, PC45T12, PC35, and PC55. Figure [9] also shows that columns PT10 and PT12 had similar compressed steel strains up to the failure load of reference column P1. Probably due to damages caused by pre-loading, these strengthened columns failed to show smaller strains. Strains on the compressed reinforcement of column P2 are not shown in the graph due to reading problems. However, it seems reasonably to believe that this column behaves similarly to column P1, as was observed in all the other graphs.

3.5 Concrete strains

The graph in Figure [10] shows the load-strain curve of the compressed concrete of all the columns tested. Columns P1, P2, and PT12 had similar strains up to the 100 kN load; as for higher loads, the strains of column PT12 decrease in relation to the other two, which remain similar up to the failure load of column P1.
Columns PC35 and PC55, strengthened on the compressed face, had the lowest maximum strains of compressed concrete, measuring 1.63 mm/m and 2.17 mm/m, respectively. The reference columns (P1 and P2) had a maximum strain of compressed concrete of 2.22 mm/m and 4.69 mm/m, respectively.
Columns PT10 and PT12, strengthened on the tension face, and PC45T10 and PC45T12, strengthened on both compressed and tension faces, reached maximum strains from 2.84 mm/m to 3.89 mm/m, respectively.

Figure 6 – Photograph of compression-strengthened columns PC35 and PC55 after failure



(a) PC35 (b) PC55

Columns PC35 and PC55 had the lowest strains, probably because of premature failure caused by the debonding of the strengthening concrete. The maximum compression strain of column PC55 (2.17 mm/m) corresponded to 72.3% of the crushing strain adopted by ACI 318M-02 [8] (3.0 mm/m) for specimens subject to combined bending and axial compression.

4. Conclusions

The static short-term loading behavior of the reinforced concrete columns strengthened by recasting with or without addition of tensile steel bars has been confirmed to be very satisfactory in

Figure 7 – Load-maximum horizontal displacement curves of columns

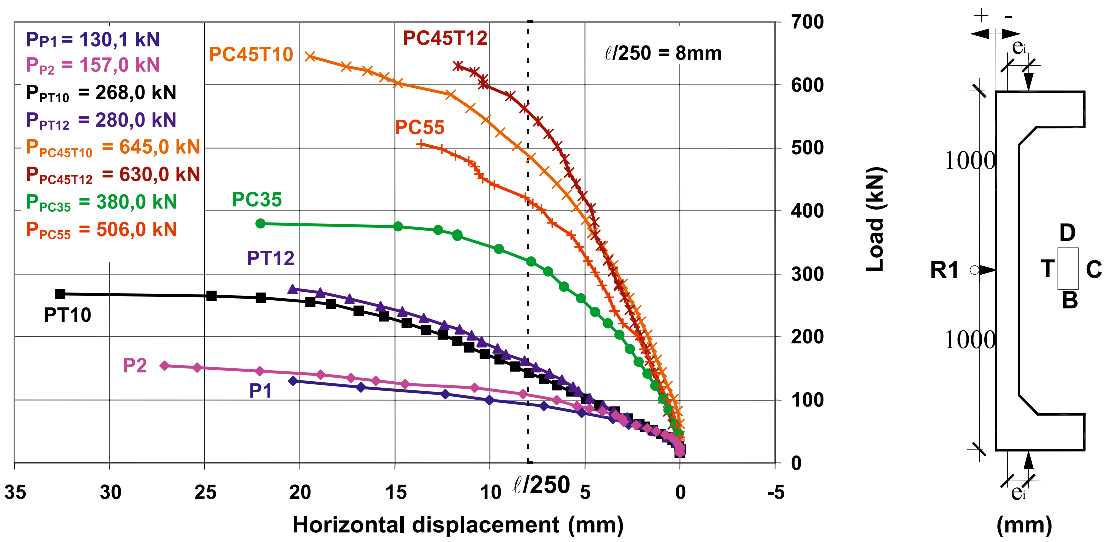
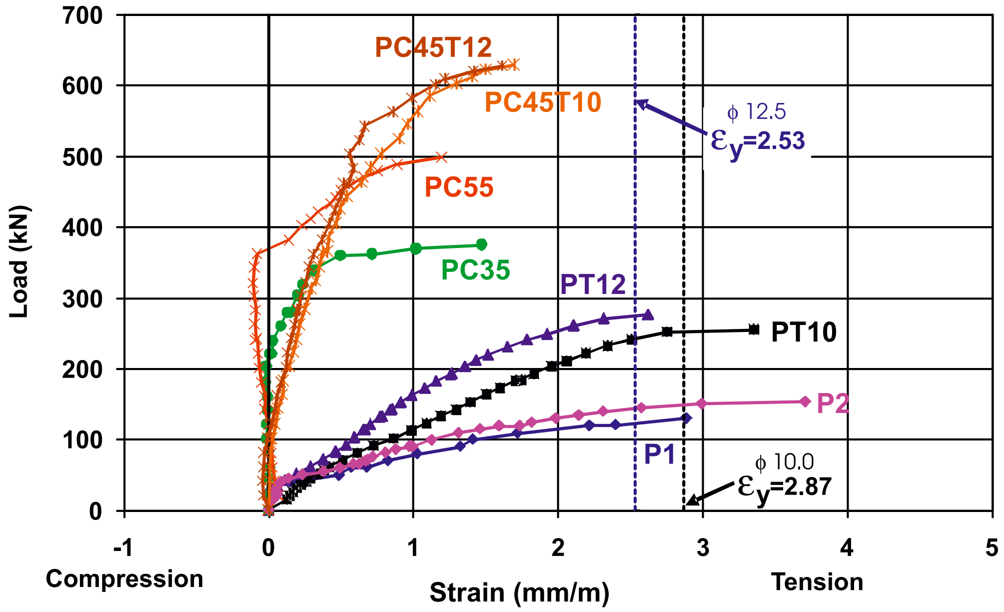


Figure 8 – Load-tensile reinforcement strain curves of columns

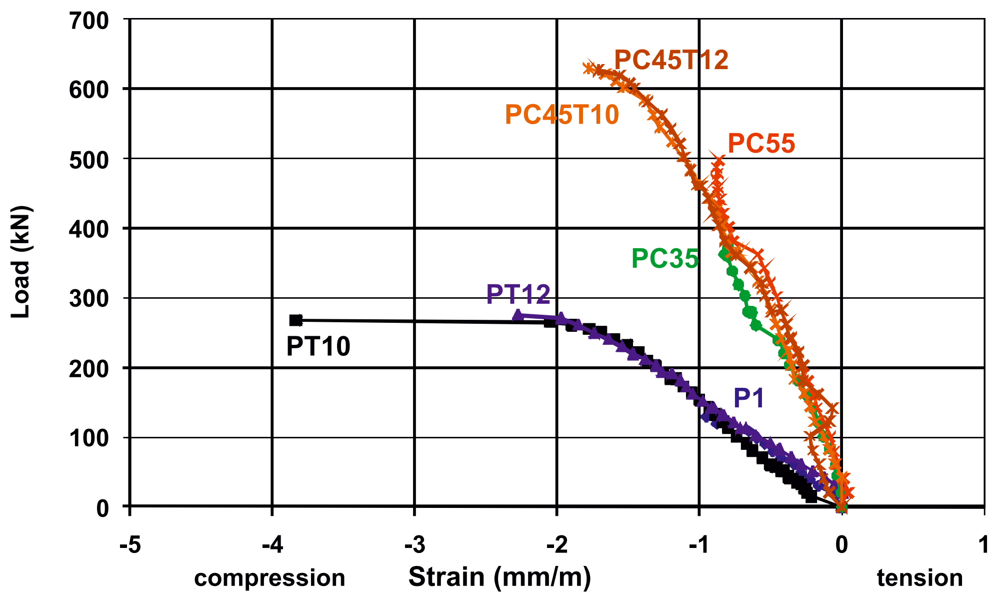


the present experimental program. However, the problem of joint cracking was a point of concern and it was understood to be of interest for the research to explore further the capability of the repair method carrying out more tests.

Since the start of testing, columns PC35, PC55, PC45T10, and PC45T12 were stiffer than column P1, thus proving the efficiency of strengthening on the reduction of horizontal displacements. The columns strengthened on compressed and tension faces showed greater stiffness (PC45T10 and PC45T12).

As regards reference columns (P1 and P2) and columns strength-

Figure 9 – Load-compressed reinforcement strain curves of columns



ened only on the tension face (PT10 and PT12), the steel is subjected to tension from the start of testing. However, the reinforcement only begins to be effectively tensioned at 19% of the failure load of columns PC45T10 and PC45T12 and at 63% and 73% of the respective failure loads of columns PC35 and PC55. One of the contributing factors is the fact that columns PC35 and PC55 had reduced initial eccentricities.

Columns strengthened on the tension face (PT10 and PT12) and on both compressed and tension faces simultaneously (PC45T10 and PC45T12) did not reveal signs of debonding of the strengthening concrete. Even though, the difference of the reinforcement rate placed on the strengthening of these columns' tension faces reached 36%, the difference in failure load was less than 5%. Therefore, an increase of the reinforcement ratio on the tension face may not produce a considerable load gain because of the possibility of concrete crushing or of an eventual debonding.

Despite concrete debonding, columns strengthened on the compressed face had a higher strength gain than those strengthened on the tension face. Considering the tension reinforcement ratio used in these specimens, these results suggest a significant increase in load-bearing capacity when the strengthening occurs on the column's compressed face, thus leading to a reduction of initial load eccentricity.

Self-compacting concrete as a strengthening material proved to be satisfactory, shaping the section of specimens without segregation or faults. As to adherence, this material worked together with the substrate, thus increasing the load-bearing capacity of the strengthened specimens. Debonding only occurred on columns PC35 and PC55, which were strengthened solely on the compressed face.

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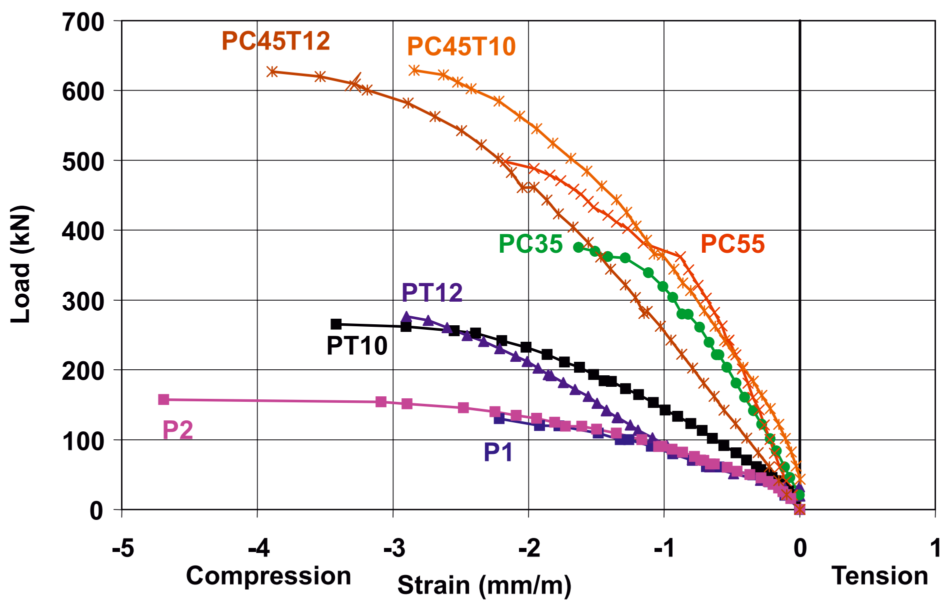
Figure 10 – Load-concrete strain curves on the compressed face of columns



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