Innovative Technique for Seismic Upgrade of RC Square Columns

by A. Prota, G. Manfredi, A. Balsamo, A. Nanni, and E. Cosenza

Synopsis: The preliminary results of an experimental investigation on under-designed RC square columns are presented in the paper. The seismic upgrade was achieved by combining steel spikes and GFRP laminates. Two parameters are investigated: the lap splice of the longitudinal steel reinforcement and the level of axial load. A comparison between as-built and strengthened columns is presented in terms of strength and ductility. The shear-top displacement relationships of strengthened columns are analyzed to assess the influence on the global performance of the lap splice. This preliminary analysis confirms that the proposed solution for the seismic strengthening of under-designed columns is very effective when it is necessary to relocalize the potential plastic hinges of columns by increasing their flexural strength. The obtained results will represent the basis for developing design criteria for the strengthening of similar interventions and will represent a reference for the calibration of a model of the strengthened column.

Keywords: column; ductility; FRP; smooth bars; strength hierarchy

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INTRODUCTION

Many existing reinforced concrete (RC) structures that are nowadays located in seismic zones have been designed about 40-50 years ago in order to withstand only gravity loads. The upgrade of their seismic performances represents an important issue that involves economic and social aspects in different areas of the world. These RC structures designed without seismic provisions are often characterized by an unsatisfactory structural behavior due to low available ductility and by a weak column-strong beam construction that, under a seismic event, yields most likely to the formation of local hinges in the columns. This failure mode represent the lower bound of the strength hierarchy because it is characterized by brittle and catastrophic structural crisis. In fact, the columns have minimum cross-sectional dimensions and their longitudinal steel reinforcement, typically smooth bars, is inadequate and has lap splices; in addition, size and spacing of the ties is often not appropriate thus the required level of confinement is not guaranteed. All these aspects can cause the collapse of the column end, resulting in crushing of the not confined concrete, instability of the steel reinforcing bars in compression and pull out of those in tension.

Different techniques can be selected to upgrade underdesigned columns. Reinforced concrete jacketing, steel profile jacketing and steel encasement have been widely used in the past. All of them were characterized by disadvantages related to constructability (i.e., difficulty of ensuring perfect bond and collaboration between old and new parts, loss of space, construction time and high impact on building functions) and durability issues; in the case of reinforcing concrete jacketing, significant mass increase could also be generated. Innovative techniques based on FRP materials have become valid alternatives to those solutions; along with high structural effectiveness, composite materials are light and easy to install, their application does not imply loss of space and, in some cases, it can be performed without interrupting the use of the structure.

Laboratory experiments have confirmed that FRP laminates can significantly improve the seismic performance of RC columns. CFRP strips were used by Ye et al. (2001) to confine square columns; tests were conducted under an axial load ratio of 0.48. Iacobucci et al. (2002) investigated the behavior of columns simulating members typical of multi-storey structures and designed with non-seismic provisions; the columns were wrapped using Carbon FRP (CFRP) laminates and the axial load ratio ranged between 0.33 and 0.56. The effectiveness of CFRP confinement to improve the seismic performance of rectangular underdesigned columns was assessed by Shaheen et al. (2003); the confinement provided by a continuous laminate was compared to that given by discountinuous strips. Bousias et al. (2004) investigated the seismic behavior of rectangular underdesigned columns with axial load ratios ranging between 0.34 and 0.40; the columns were wrapped with either CFRp or GFRP and the effect of corrosion was also studied. The opportunity of using FRP to repair damaged columns has been also verified. Ilki and Kumbasar (2001) tested the effectiveness of longitudinal and transverse CFRP laminates to restore the performance of damaged square columns with axial load ratios ranging between 0.05 and 0.20. Chang et al. (2004) tested 2/5 scale rectangular columns repaired using CFRP confinement and the pseudo-dynamic tests confirmed that the original seismic performance could be recovered after the FRP repair.

Tests have been also performed to assess the possibility of preventing the failure of the column due to lap splices of the longitudinal steel bars using FRP. Chung et al. (2002) demonstrated that Glass FRP (GFRP) confinement could be able to avoid the lap splice failure on circular bridge columns. Haroun et al. (2002) analyzed both circular and rectangular half-scale columns confined with either CFRP or GFRP laminates; they found that the FRP confinement could be successful to enhance the ductility of circular columns with insufficient lap splice length, but could not be able to inhibit the lap splice slippage on rectangular columns. This was also confirmed by tests on large-scale square columns confined with CFRP performed by Saatcioglu and Elnabelsy (2001). Kono et al. (2004) analyzed the influence of CFRP confinement on the bond-slip relationship of steel reinforcing bars. In the present paper, the preliminary results of an experimental campaign aiming at assessing the possibility of combining steel fibers and GFRP laminates for the seismic upgrade of underdesigned RC columns are outlined. The analyzed parameters are the axial load ratio and the detail of the lap splice above the construction joint. Both monotonic and cyclic tests have been performed; only those monotonic are herein discussed.

TEST SPECIMENS

A total of 8 column specimens were constructed and tested under monotonic lateral load. All had the same square cross-section with side equal to 300 mm and were internally reinforced using smooth steel bars. The specimens were characterized by height of 2.0 m above the footing 0.60 m deep. 8-mm diameter ties spaced at 100 mm on center were placed along the height; rules typical of old construction were followed for the first tie above the footing (placed at 50 mm from the column-footing interface) as well as for the geometry of the hooks. Two different layouts of the longitudinal reinforcement were realized. In type C columns, each of the three 12-mm diameter longitudinal bars disposed on each side of the cross-section had no lap splice from the footing (Figure 1); type LP columns had instead lap splices of the longitudinal reinforcement as depicted in Figure 2. Tests on cylinders taken during specimen construction provided an average cylindrical concrete strength of 24.9 MPa; the mechanical characterization of the used smooth bars provided a yield strength of 358 MPa and 327 MPa, a maximum strength of 449 MPa and 439 MPa, and a strain corresponding to the ultimate strength of 21.5% and 23.1%, for 8 mm-diameter and 12 mm-diameter smooth bars, respectively.

STRENGTHENING CONFIGURATION

The design of the strengthening of the columns could not be done without considering the consequences of the column upgrade on the global performance of the structure. When operating on underdesigned structures, the local upgrade with composites should aim at improving the global deformation capacity of the structure. One way to reach this goal is to relocalize the potential plastic hinges; this means to establish a correct hierarchy of strength, that is a key criterion in the seismic design of new structures as suggested by seismic codes of Europe, USA, New Zealand and Mexico. Therefore, the strategy is the following: by increasing the strength of some elements (i.e., columns) it is achieved that the failure of others (i.e., beams) occurs before and then prevents that of the upgraded members. This allows protecting those elements whose failure could be critical from a global standpoint and then improving the seismic behavior of the structure.

Tests performed on RC columns and numerical analyses (Grasso et al., 2003) have demonstrated that the FRP wrapping can provide a significant benefit in terms of ductility of the confined cross-section, but the strength increase is negligible for axial load ratios that characterize typical columns of buildings or bridges. Since the goal of the strengthening was to modify the strength hierarchy, it was necessary to design a strengthening scheme that could allow increasing also the strength of the column. An innovative system was then proposed based on the combination of steel spikes and GFRP laminates. The steel spikes were made of 3x2 steel cords (Hardwire 2002), each of them being obtained by twisting 5 individual zinc coated wires together (Figure 3); they were realized by cutting a roll of steel cords (Figure 4-a). The density of the 3X2 tape used in this research program consists of 87 cords per mm, which is considered high-density tape. The steel cords have an ultimate tensile strength of 3070 MPa, Young modulus of 184 GPa and ultimate strain equal to 0.017. A two-component thixotropic epoxy resin

Adesilex PG1 (Mapei 2003) was used to impregnate and bond the steel tape to the concrete substrate. GFRP uniaxial laminates having a density of 900 gr/m^2 were used (Figure 4-b). The supplier provides the following properties of these laminates: ultimate tensile strength equal to 1370 MPa, Young modulus equal to 65.6 GPa and ultimate strain equal to 0.021.

The strengthening sequence can be summarized as follows. Two 18-mm diameter holes were realized on each side of the column (Figure 5-a) and, for the wrapping length, the corners were rounded and the surface was sandblasted. Each hole, 300 mm deep, was cleaned and consolidated with primer (Figure 5-b); primer and putty were applied to the portion of the column to be wrapped with GFRP laminates. Strips 700 mm long and 70 mm wide were cut from the roll of steel cords; out of the 700 mm, 300 mm of each strip were twisted by hands and embedded into a Adesilex PG1 container. Adesilex PG1 was injected in every hole and then each steel spike was inserted (Figure 6-a). After it was inserted into the hole, the portion of the spike 400 mm long was then bonded to the column surface (Figure 6-b). Once the steel spikes were installed, the column was wrapped with two plies of GFRP laminates (Figure 7). The cross-section of the strengthened column is shown in Figure 8; Figures 8-9 depict the geometry of the strengthening scheme.

EXPERIMENTAL PROGRAM

A total of 8 columns were tested, 4 as-built and 4 strengthened. Within each series of 4 specimens, two parameters were studied: the axial load ratio and the lap splice of the longitudinal steel reinforcement. The two selected axial loads were 270 kN and 540 kN corresponding to axial load ratios of 0.12 and 0.24, respectively. For each of those two axial load ratios, one type C and one type LP columns were tested. Each specimen is denoted in the following by a letter (C for columns without lap splice and LP for those with lap splice) followed by a number that indicates the value of the axial load (270 kn or 540 kN). The test setup is shown in Figure 11. Two post-tensioned bars were used to connect the footing of each specimen to the strong floor. Even though the post-tensioning of the bars was computed in order to avoid the sliding of the specimen during test, lateral restraints were also provided on the short sides of the footing in order make the test system more safe. The vertical and horizontal hydraulic actuators were put in place; then, the axial load was applied. Once the column was under the fixed level of axial load, the lateral load started to act. Tests were performed in a displacement control mode. The control system assured that the axial load was constant during the entire test. Draw wire transducers measured the displacements of the column at the top and at the height of the horizontal actuator; linear variable displacement transducers (LVDT) measured compressive and tensile deformations on the sides of the column on a gage length of 40 mm. A view of a column prior to testing is shown in Figure 12.

PRELIMINARY EXPERIMENTAL RESULTS

A detailed analysis of the experimental performance of the as-built columns is done elsewere (Verderame et al., 2004). The strengthened columns showed significant strength increases compared with those as-built. Table 1 summarizes the experimental outcomes for both as-built and strengthened columns in terms of shear corresponding to longitudinal steel bars yielding, F_y, maximum shear force, F_{max}, drift corresponding to F_y, d_{Fy}, drift corresponding to a shear force equal to 90% of F_{max} on the descending branch, $d_{F90\%}$, and ductility index, δ, computed as ratio $d_{F90\%}/d_{Fy}$. In terms of strength, the results show an average increase of 54% for the columns subjected to axial load of 270 kN and 33% for those whose axial load was 540 kN. The drift at 90% of the maximum shear force on the descending branch was less than that given by the as-built columns for the low axial load, whereas it almost doubled when the axial load was 540 kN. In terms of ductility index, the strengthened columns provided in all cases values lower than the corresponding as-built members. In terms of global behavior of the strengthened columns expressed in terms of shear-top displacement relationships, it appears that for axial load of 270 kN the trend was not very influenced by the lap splice of the longitudinal reinforcement Figure 13. A different situation was observed for columns under 540 kN (Figure 14): the specimen LP-540 showed a higher stiffness compared with C-540. This could be due to the different failure mode: in the case of C-540 it was characterized by a reduced crack at the footing interface and by another significant crack that opened at the height where the spike was cut (Figure 15); the crack pattern of LP-540 until failure showed only one significant crack at the footing interface Figure 16. In terms of strength, for both axial load levels type LP columns had a slightly higher strength compared with the corresponding of type C; this is consistent with the fact that for the length of the lap splice the amount of steel longitudinal reinforcement was double.

CONCLUSIONS

The paper presented the preliminary outcomes of an experimental analysis concerning monotonic and cyclic behaviour of under-designed RC square columns. An innovative technique based on the combination of steel spikes as flexural reinforcement and GFRP laminates as external confinement was validated in the laboratory. The comparison between as-built and strengthened columns provides strength increases ranging between 33% and 54% with increase also of the drift corresponding to maximum shear force. The obtained results allow also assessing the influence of the lap splice of the longitudinal reinforcement on the global behaviour of the column. The obtained results will be used as a reference for the calibration of a model of the strengthened column and they will be also the basis for the development of design criteria that engineers could use to design similar strengthening interventions.

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Table 1 – Summary of Experimental Results

	As-built					Strengthened				
	F_y (kN)	F_{max} (kN)	d_{Fy} (%)	$d_{F90\%}$ (%)	δ	F_y (kN)	F_{max} (kN)	d_{Fy} (%)	$d_{F90\%}$ (%)	δ
C-270	41.44	42.38	0.96	5.73	5.97	60.69	61.87	1.26	4.66	3.70
LP-270	37.04	39.24	0.66	5.47	8.29	61.77	63.57	1.59	4.53	2.85
C-540	60.46	63.21	0.89	3.05	3.43	80.06	82.79	1.69	5.05	2.99
LP-540	58.26	62.61	0.71	3.18	4.48	83.19	84.40	1.58	6.16	3.89

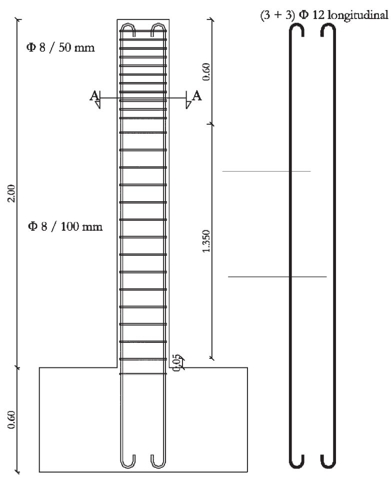


Figure 1 – Geometry and steel reinforcement layout of type C columns

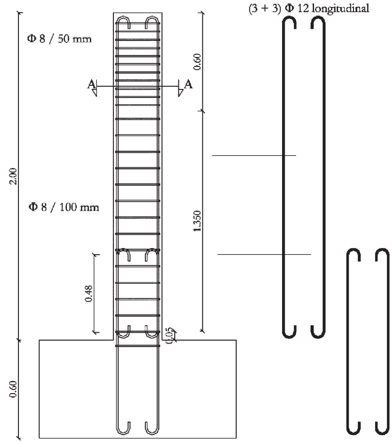


Figure 2 — Geometry and steel reinforcement layout of type LP columns

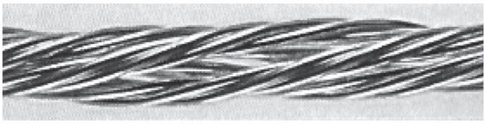


Figure 3 — 3X2 cord

(a) (b)

Figure 4 – Steel roll from which spikes were obtained (a) and cutting of the GFRP sheets (b).

(a) (b)

Figure 5 – Realization of holes (a) and consolidation of holes with primer (b)

(a) (b)

Figure 6 – Steel spike inserted into the pre-injected hole (a) and steel spike installed on the column (b)

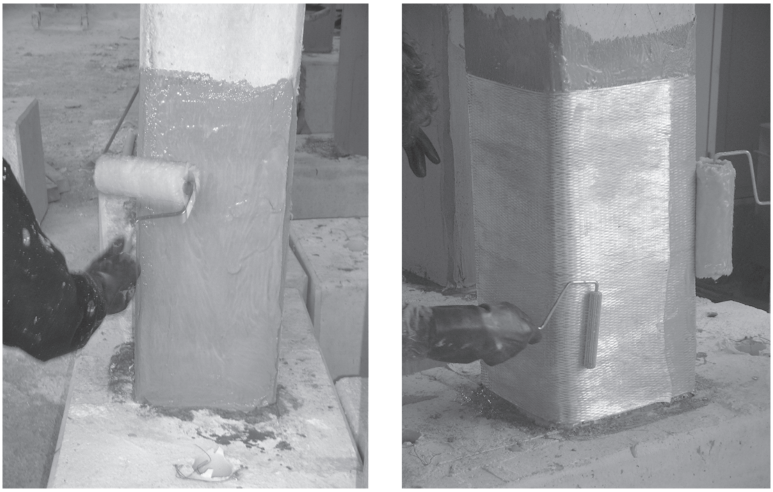


Figure 7 – Installation of GFRP laminates after placement of the steel spikes

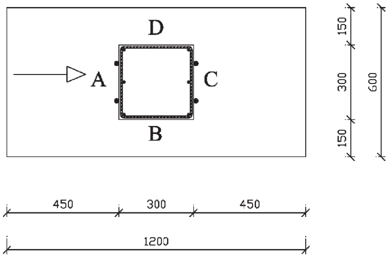


Figure 8 – Cross-section of the strengthened column

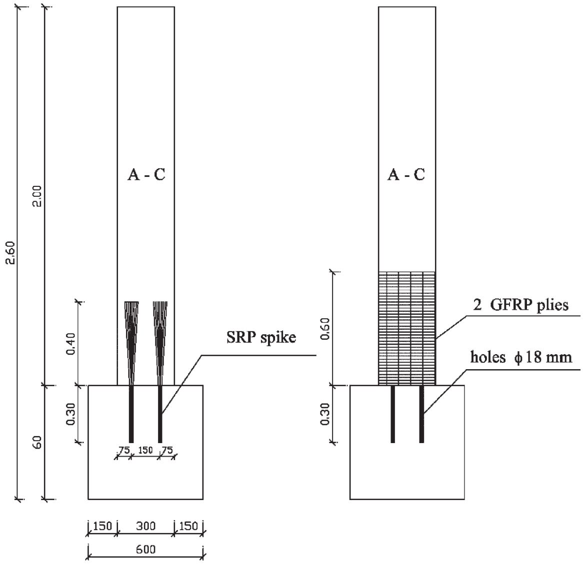


Figure 9 – Cross-section parallel to sides A and C: steel spikes (a) and final strengthening configuration (b)

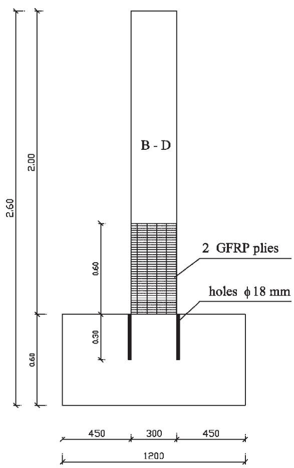


Figure 10 — Cross-section parallel to sides B and D: final strengthening configuration

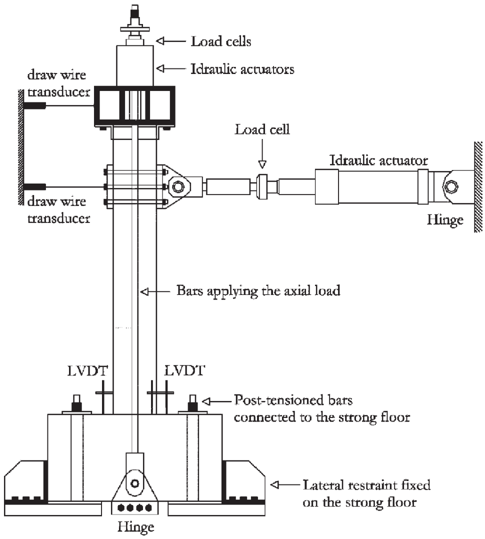


Figure 11 — Test setup

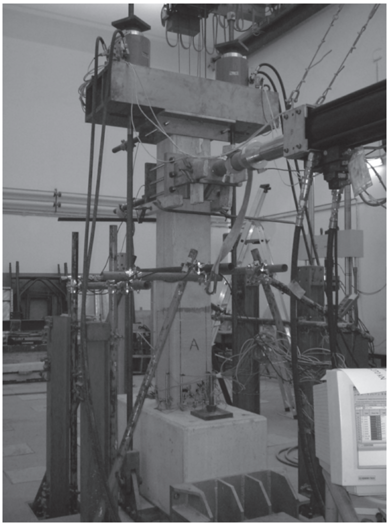


Figure 12 – View of a column prior to testing

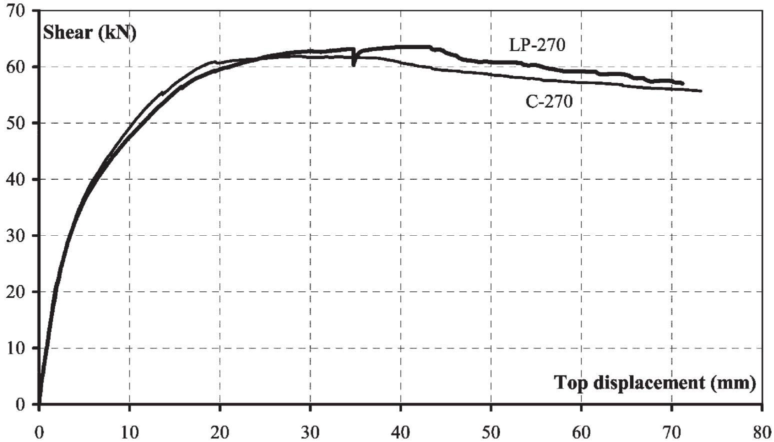


Figure 13 – Shear-top displacement curves for C-270 and LP-270 columns

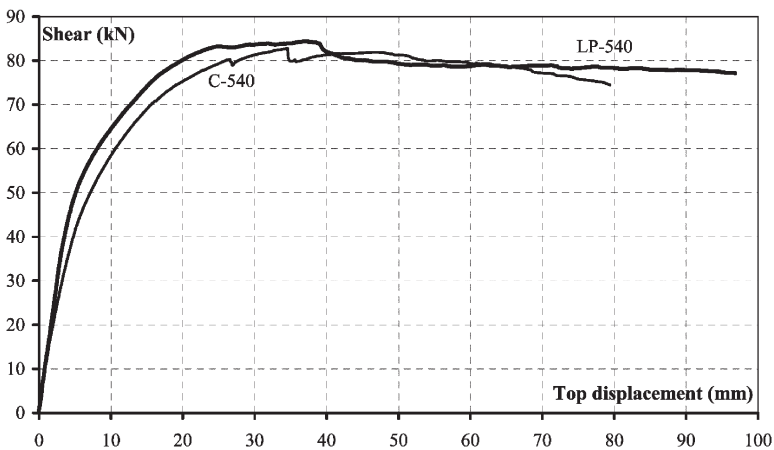


Figure 14 – Shear-top displacement curves for C-540 and LP-540 columns

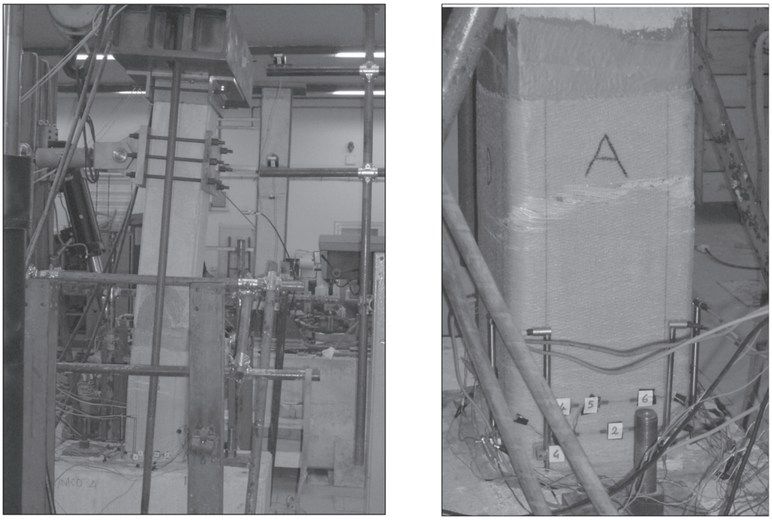
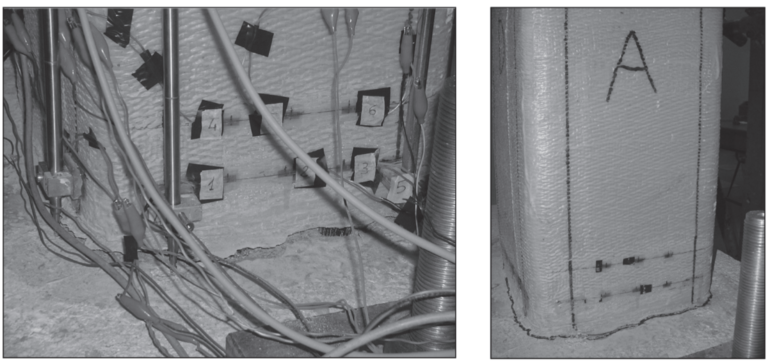


Figure 15 – Failure mode of C-540 column



(a) (b)

Figure 16 – Column LP-540 at end of the test: crack at column-footing interface (a) and view of side A (b)