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Stathis Bousias [a], Alexis-Loukas Spathis [a] & Michael N. Fardis [a]
[a] Structures Laboratory, Department of Civil Engineering, University of Patras, Patras, Greece
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Seismic Retrofitting of Columns with Lap Spliced Smooth Bars Through FRP or Concrete Jackets

STATHIS BOUSIAS, ALEXIS-LOUKAS SPATHIS, and MICHAEL N. FARDIS

Structures Laboratory, Department of Civil Engineering, University of Patras, Patras, Greece

The effectiveness of RC jacketing or FRP wrapping for seismic retrofitting of rectangular columns having smooth (plain) bars with 180° hooks lap-spliced at floor level is experimentally investigated. The relatively low deformation capacity and energy dissipation of five unretrofitted columns is found not to depend on lap length, if lapping is not less than 15 bar-diameters. Six columns cyclically tested up to ultimate deformation after RC concrete jacketing demonstrate force and deformation capacity and energy dissipation sufficient for earthquake resistance, regardless of the presence or length of lap splicing in the original column. Another ten columns cyclically tested to ultimate deformation after wrapping of the plastic hinge region with CFRP show that FRP wrapping of the splice region is more effective than concrete jackets for enhancement of the deformation and energy dissipation capacity of old-type columns with smooth bars lap-spliced at floor level, provided that wrapping extends over the member length sufficiently to preclude plastic hinging and early member failure outside the FRP-wrapped length of the column.

Keywords Concrete Jackets; Cyclic Deformation Capacity; Cyclic Tests; Frp Jackets; Lap Splices; Seismic Rehabilitation; Seismic Retrofitting; Seismic Upgrading

1. Introduction

The majority of the concrete building stock in the seismic parts of the world has been constructed before enforcement of modern seismic design codes and is, thus, inherently vulnerable to earthquakes. Smooth (plain) bars with hooked ends were commonly used in the past — in European countries even during the 1970's or even later. Moreover, in columns, such bars were lap-spliced at column ends (typically at floor levels). As this type of deficiency is often the weak link, remedying it is essential for seismic upgrading of old concrete buildings.

Concrete jacketing is still the method of choice for seismic rehabilitation of members in existing concrete buildings, especially in seismic-prone European countries. This is due to the cost-effectiveness of the technique, owing to:

- The familiarity of engineers and the construction industry alike with the field application of structural concrete.
- The suitability of concrete jacketing for repair of severe seismic damage (including concrete crushing, or buckling of bars and fracture of stirrups), in the same operation as casting or shotcreting the jacket.

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Address correspondence to Stathis Bousias, Structures Laboratory, Department of Civil Engineering, University of Patras, Patras 26500, Greece; E-mail: sbousias@upatras.gr

- The versatility and shape-adaptability of reinforced concrete to fully encapsulate existing members and joints and to provide structural continuity between different components (between a joint and the adjoining members, between members in adjacent storeys, etc.).
- The ability of concrete jackets to have, when suitably reinforced, multiple effects. They can enhance at the same time stiffness and flexural resistance (through the increased size of the cross section and the added longitudinal reinforcement that easily extends beyond the member end, into and through the joints), as well as shear resistance, deformation capacity, and anchorage and continuity of reinforcement in development or splicing zones (due to the added transverse reinforcement, that works for shear, confinement, and against buckling of longitudinal bars).
- The reduction of global seismic displacements and the deformation demands in all members — possibly below their deformation capacities — due to the increased stiffness of the jacketed members.
- The effectiveness of concrete jackets, constructed continuous from story-to-story around the joint, to remedy weak column/strong beam cases and/or deficient beam-column joints.

Considering the benefits and popularity of concrete jacketing, the scarcity of cyclic tests on RC-jacketed members, especially on columns having short lap-splicing of vertical bars in the plastic hinge region and/or smooth (plain) vertical bars, is striking.

Unlike concrete jackets, wrapping concrete columns with fiber-reinforced-polymers (FRP) is under intensive study worldwide, owing to the advantages it offers over RC jacketing, notably:

- the smaller final column section, if floor area is at a premium;
- less disruption of occupancy;
- little pollution with dust, debris and noise;
- better worker safety, etc.

As FRP jackets normally do not extend beyond the end of the existing member and into joint, they do not increase flexural resistance and stiffness, but improve only shear strength, deformation capacity, and splicing or anchorage of longitudinal reinforcement.

FRP retrofitting of circular or rectangular columns with inadequate lap-splices of ribbed (deformed) bars with straight ends has been studied by Chang *et al*. [2001], Elnabelsy and Saatcioglu [2004], Harries *et al*. [2003], Haroun *et al*. [2001], Laplace *et al*. [2005], Ma *et al*. [1997, 2000], Restrepo, *et al*. [1998], Saadatmanesh *et al*. [1997], Saadatmanesh *et al*. [1997], Saatcioglu and Elnabelsy [2001], Schlick and Brena [2004], and Xiao and Ma [1997].

By contrast, there is almost complete lack of published information on the cyclic behavior of FRP-retrofitted columns with smooth (plain) bars and hooked ends, with or without lap splices. Cyclic test results on unretrofitted columns with smooth bars and hooked ends, lap-spliced or continuous, are also scarce, despite the widespread use of this type of bars in some countries in the past. This article contributes to filling this gap via experimental work.

The article presents and discusses the results of an experimental program on rehabilitation of rectangular RC columns with smooth (plain) vertical bars lap-spliced at floor level through concrete or FRP jackets. In addition to the type of jacket, the parameters studied were: (a) the length of lapping; (b) the number of FRP layers; and (c) the length of application of FRP wrapping. As concrete jacketing is the technique of choice for the repair/strengthening of seismically damaged columns, the article investigates also the impact of previous unrepaired damage on the effectiveness of concrete jackets.

2. Test Specimens, Materials, Test Set-Up and Testing Program

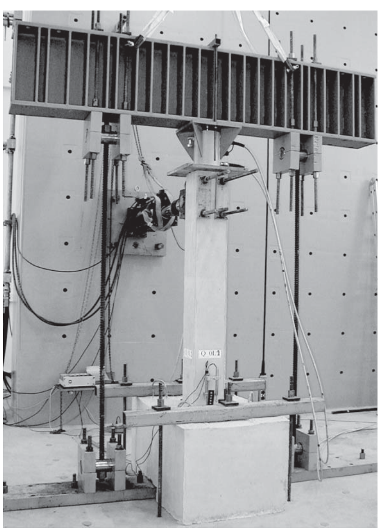
The original column specimens had dimensions, reinforcement detailing, and materials typical of old RC buildings without detailing for earthquake resistance. It had (see Fig. 1(b)) a 250 mm square section and was reinforced with 4, 14 mm smooth (plain) corner bars having yield stress 313 MPa and tensile strength 442 MPa (average from three coupons). To represent past practice, the transverse reinforcement of the original column consisted of smooth 8 mm stirrups at 200 mm centers, anchored by a 135° hook at one end and a 90° hook at the other (obviously, such widely spaced stirrups are not expected to be very effective for confinement or clamping of lapped bars). The stirrup yield stress and tensile strength were 425 MPa and 596 MPa, respectively. The value of the concrete cylindrical strength of the original column at testing, f_c, is listed in Tables 1 to 3.

The unretrofitted specimens are given the notation Q_0Li, where *Li* denotes reinforcement lapping (L1 for 15-bar diameters lapping and L2 for 25-bar diameters lapping). Information on the five columns tested unretrofitted is given in Table 1.

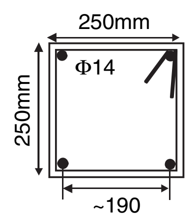
Retrofitted columns are denoted as Q_*XYZ*, where *X* signifies the type of jacket used (RC for reinforced concrete and Pj for j = 2 or 4 layers of FRP), *Y* the lapping length (nothing for continuous reinforcement, L1 or L2 for 15- or 25-bar diameters, respectively), and *Z* the height of FRP-wrapping (H1 for 300 mm and H2 for 600 mm). Suffix *pd*, when used, denotes that the specimen was damaged before retrofitting. Table 2 provides details for the six columns tested after RC-jacketing, plus a monolithic control specimen having the same reinforcement as the concrete jacket applied to the other columns. Table 3 refers to the ten columns tested after FRP-wrapping.

The column was fixed into a heavily reinforced 0.6 m-deep footing (Fig. 1(a)). The footing was prestressed to the laboratory strong floor to avoid footing rotation. This was verified by the measurements of dial gages attached to the footing checking that the displacement relative to the strong floor was nil. Each column was tested as a simple cantilever. Its shear span, measured from the top of the footing to the point of

(a)



(b)



(c)

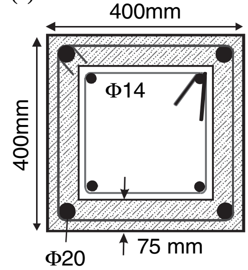


FIGURE 1 Test setup (a) and cross section of original (b) and of RC-jacketed column (c) — dimensions are in mm.

TABLE 1 Unretrofitted columns: test parameters and key results

Specimen (1)	lap length (2)	effective depth (mm) (3)	f_c (MPa) (4)	$\nu = N/bhf_c$ (5)	yield moment, M_y (kNm) (6)	drift at yielding θ_y (%) (7)	yield curvature φ_y (1/m) (8)	fixed-end rotation at yielding (rad) (9)	ultimate drift, θ_u (%) (10)	ultimate curvature φ_u (1/m) (11)	fixed-end rotation at ultimate (rad) (12)	Main features of the behavior and of failure mode (13)
Q-0	–	210	27.0	0.44	66	0.8	0.026	0.0015	2.2	0.085	0.0023	heavy concrete crushing up to 300 mm from the base
Q-0L1	$15d_{bL}$	220	30.3	0.41	72.5	1.05	0.029	0.001	2.5	> 0.095†	> 0.0038†	early cracking and then spalling of corners all along lapping; bar buckling and spalling of corners mainly *above lap splice*
Q-0L2	$25d_{bL}$	220	30.3	0.42	72.5	0.95	0.029	0.0013	2.2	0.135	0.0088	corners spalled all along splice; bar buckling tendency along the splice
Q-0L1a	$15d_{bL}$	220	28.1	0.63	70	0.8	0.009	0.0006	1.0	—‡†	—‡†	pre-damaged specimen for Q-RCL1pd; failure mode as in Q-0L1
Q-0L2a	$25d_{bL}$	220	28.1	0.57	83	0.9	0.017	0.008	1.35	—‡†	—‡†	pre-damaged specimen for Q-RCL2pd, failure mode as in Q-0L1

†base was not critical; failure occurred outside instrumented region.
‡measurements insufficient for estimation of curvature and fixed-end rotation.

TABLE 2 RC-jacketed columns: test parameters and key results

Specimen	lap length	effective depth (mm)	concrete strength, f_c (MPa) original column	concrete strength, f_c (MPa) jacket	$\nu = N/bhf_c$ jacketed column	yield moment M_y (kNm)	drift at yieldingy θ_y (%)	yield curvature φ_y (1/m)	fixed-end rotation at yielding (rad)	ultimate drift, θ_u (%)	ultimate curvature φ_u (1/m)	fixed-end rotation at ultimate (rad)	Main features of the behavior and of the failure mode
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Q-RCM	–	350	30.6	–	0.18	251	1.0	0.029	0.001	5.3	—‡	—‡	concrete crushed and all four bars buckled at base; one bar ruptured
Q-RC	–	355	26.3	55.3	0.08	233	1.3	0.029	0.0008	5.3	0.35	0.0250	severe disintegration near base; lower-most tie opened; two jacket bars and one interior bar buckled (old column); one bar ruptured
Q-RCpd	–	355	23.1	24.1	0.168	243	1.25	0.028	0.0008	5.3	0.135	0.0063	heavy bond splitting/spalling all along the corner bars

(*Continued*)

TABLE 2 (*Continued*)

Specimen (1)	lap length (2)	effective depth (mm) (3)	concrete strength, f_c (MPa) original column (4)	jacket (5)	$\nu = N/bhf_c$ jacketed column (6)	yield moment M_y (kNm) (7)	drift at yieldingy θ_y (%) (8)	yield curvature φ_y (1/m) (9)	fixed-end rotation at yielding (rad) (10)	ultimate drift, θ_u (%) (11)	ultimate curvature φ_u (1/m) (12)	fixed-end rotation at ultimate (rad) (13)	Main features of the behavior and of the failure mode (14)
Q-RCL1	$15d_{bL}$	360	27.5	55.3	0.085	213	1.15	0.030	0.0008	5.6	0.34	0.0225	full disintegration near base; partial height bond splitting/spalling along all corner bars
Q-RCL2	$25d_{bL}$	360	25.6	55.3	0.085	217.5	1.0	0.030	0.0013	5.3	0.29	0.0225	bond splitting/ spalling all along corner bars; diagonal cracks
Q-RCL1pd	$15d_{bL}$	360	28.1	28.7	0.16	204	1.0	0.027	0.0008	4.4	0.21	0.0087	full disintegration up to 500 mm from base; all four jacket bars buckled
Q-RCL2pd	$25d_{bL}$	360	28.1	28.7	0.175	245	1.1	0.028	0.0013	5.3	—‡	—‡	buckling of all jacket bars and of interior bars (old column)

‡measurements insufficient for estimation of curvature and fixed-end rotation.

TABLE 3 FRP-jacketed columns: test parameters and key results

Specimen (1)	lap length (2)	effective depth (mm) (3)	layers and height of FRP jacket (4)	concrete strength, f_c (MPa) (5)	$\nu = N/bhf_c$ (6)	yield moment M_y (kNm) (7)	drift at yielding θ_y (%) (8)	yield curvature φ_y (1/m) (9)	fixed-end rotation at yielding (rad) (10)	ultimate drift, θ_u (%) (11)	ultimate curvature φ_u (1/m) (12)	fixed-end rotation at ultimate (rad) (13)	Main features of behavior and failure mode (14)
Q-P4H2	–	215	four–0.6m	28.2	0.44	77.0	1.15	0.043	0.001	θ_u>6.9	>0.30	>0.0063	no visible damage; tests ended before reaching conventional ultimate drift
Q-P4H1	–	215	four–0.3m	28.1	0.45	78.3	1.25	0.041	0.0007	θ_u>6.6	>0.47	>0.0275	
Q-P4H1a	–	215	four–0.3m	28.2	0.45	62.4	0.9	0.013	0.0005	2.2†	0.035†	0.0019†	no FRP fracture; concrete crushing and bar buckling *above FRP*
Q-P2H2	–	215	two–0.6m	28.1	0.45	74.5	0.95	0.024	0.001	4.7	0.20	0.0050	FRP fracture over lowest 170 mm; concrete dis-integration and bar buck-ling inside

(*Continued*)

TABLE 3 (*Continued*)

Specimen (1)	lap length (2)	effective depth (mm) (3)	layers and height of FRP jacket (4)	concrete strength, f_c (MPa) (5)	$\nu = N/bhf_c$ (6)	yield moment M_y (kNm) (7)	drift at yielding θ_y (%) (8)	yield curvature φ_y (1/m) (9)	fixed-end rotation at yielding (rad) (10)	ultimate drift, θ_u (%) (11)	ultimate curvature φ_u (1/m) (12)	fixed-end rotation at ultimate (rad) (13)	Main features of behavior and failure mode (14)
Q-P2H1	–	215	two–0.3m	28.2	0.40	77.4	1.0	0.033	0.001	5.0	0.335	0.0213	FRP fracture over lower 120 mm; concrete disintegration but no bar buckling inside
Q-P4L1H2	$15d_{bL}$	220	four–0.6m	30.0	0.40	76.8	1.1	0.023	0.0005	6.8	0.52	0.035	no FRP fracture or visible damage
Q-P4L1H1	$15d_{bL}$	215	four–0.3m	27.5	0.44	82.8	1.1	0.031	0.0008	6.2	0.54	0.040	FRP fracture over lowest 20–40 mm; no visible damage inside
Q-P2L1H2	$15d_{bL}$	215	two–0.6m	30.0	0.40	75.0	1.2	0.029	0.0015	6.0	0.42	0.0275	
Q-P4L2H2	$25d_{bL}$	215	four–0.6m	30.0	0.42	83.7	1.4	0.03	0.0008	6.9	0.65	0.0525	
Q-P2L2H2	$25d_{bL}$	220	two–0.6m	30.0	0.43	79.2	1.2	0.032	0.0015	6.0	0.44	0.0325	

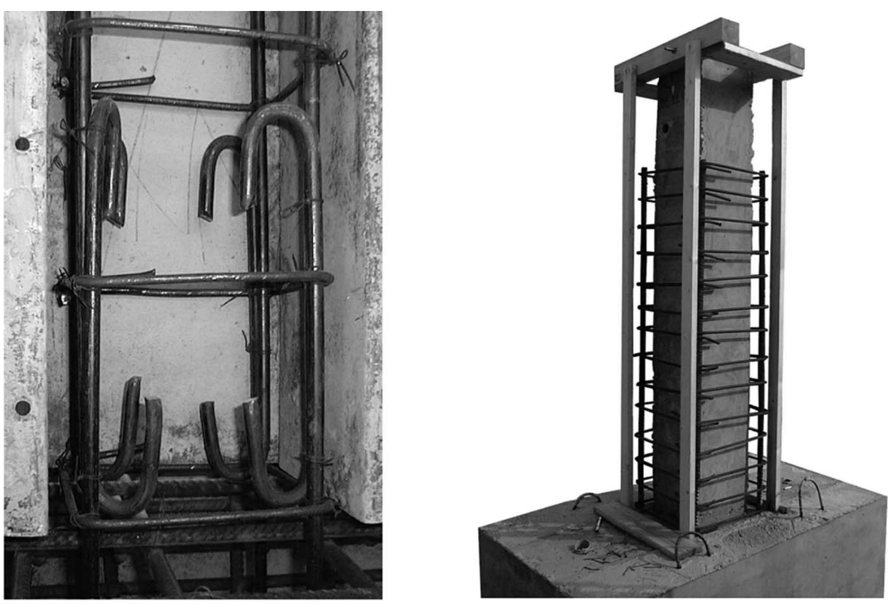
†base was not critical; failure occurred outside instrumented region.

application of the lateral load, was 1.6m and was meant to be equal to about half a typical story height.

Eight of the original columns (the ones not designated as L1 or L2 in Tables 1–3) had continuous vertical bars extending to the bottom of the footing and anchored there with 180° hook. In 13 columns, starter bars with 180° hooks were provided at the base of the original column and lap-spliced with the main bars of the column that started at the base section with a 180° hook. Lapping took place over a length of 15- or 25-bar diameters, measured to the tangent of the hook (specimens L1 or L2, respectively, in Tables 1–3, see Fig. 2(a)). The internal diameter of the hook was equal to 2.5 bar-diameters and the bar extended straight past the end of the curved part of the hook by 40 mm.

The concrete jacket of the columns in Table 2 was 75 mm thick (Fig. 1(c)). It consisted of shotcrete, except in specimen Q-RCpd, where it was cast-in-place. A 75 mm thickness was almost the minimum that could provide proper cover to the perimeter tie and allow a 135° hook at the tie end. The jacket was reinforced longitudinally with 4, 20 mm ribbed (deformed) bars with a yield stress of 487 MPa, which were embedded in the footing when the original column was cast (Fig. 2(b)). The jacket transverse reinforcement consisted of a single 10 mm perimeter tie at 100 mm centers, having yield stress 599 MPa. With the addition of the RC jacket, the cross-sectional dimension of the column became 400 mm and its shear span ratio decreased from 6.4 to 4.0. The jacket stopped about 250 mm below the point where the lateral load was applied. The cylindrical compressive strength of the jacket at the time of testing is listed in Table 2.

A monolithic column, Q-RCM, was also constructed for testing as control specimen, with the same external dimensions (400 mm square) and the same reinforcement as the jacket (4, 20 mm vertical bars, 10 mm closed ties at 100 mm centers), but without the vertical or transverse reinforcement of the original column.



(a) (b)

FIGURE 2 Laps at column base in specimen Q-0L2. (a) jacket reinforcement embedded in column base (b).

Two specimens, Q-RCL1 and Q-RCL2, with lap-spliced vertical bars in the original column, were jacketed without any special measure for connection of the jacket to the old concrete. To study the effect of initial damage, specimen Q-RCpd was jacketed after cyclic testing beyond yielding and up to almost ultimate strength. Initial damage consisted of open but narrow cracks and spalled or locally crushed concrete. This damage was left unrepaired; only spalled or crushed concrete was replaced by the jacket shotcrete. Again, no special measure was taken to connect the jacket to the old concrete. This was because Bousias *et al*. [2006] have found, through an experimental campaign of their own and a comparative study of test results in the literature, that in RC-jacketed columns without lap splices in the original column, positive measures of connection (such as a roughened interface, steel dowels, or connection of the jacket corner bars to the old ones through welded U-bars) are not essential for the full composite action of the old and the new concrete under cyclic loading. The same conclusion was reached in a comparative monotonic testing program of six jacketed columns having different types and levels of connection at the interface [Julio *et al*, 2005].

In two more jacketed specimens, Q-RCL1pd and Q-RCL2pd, the original columns were specimen Q-0L1a and Q-0L2a, respectively, previously damaged by cyclic testing beyond ultimate conditions (see column (13) in Table 1 for the description of the initial damage to these two columns). Unlike Q-RCpd and any other of the jacketed columns, Q-RCL1pd and Q-RCL2pd had the full lateral surface of the old column roughened with an electric concrete chipping hammer, until the hardened cement paste and fine aggregates were removed and coarse aggregates exposed.

Ten columns were tested after retrofitting with wraps of resin-impregnated, unidirectional, carbon-fiber-reinforced-polymer (CFRP) sheets. Each ply of CFRP had nominal thickness 0.13 mm, Elastic Modulus 230 GPa, and tensile strength 3450 MPa (failure strain 1.5%) in the (main) direction of the fibers. The CFRP sheets were attached to the column via epoxy resin, after thoroughly cleaning the column surface of loose material and rounding the corners of the section to a radius of 30 mm, to avoid stress concentrations that may lead to premature FRP rupture and to extend the confining action of the FRP at the corner over a larger concrete volume. A CFRP lap length of one full side of the cross-section was provided at the end of the wrap.

Different numbers of CFRP plies were used and over different heights from the column base. In the notation of the specimens, P2 or P4 stands for two or four CFRP layers, respectively; H1 is also used if CFRP was applied over the lower 0.3 m and H2 if it was applied over the lower 0.6 m. The FRP wrapping started at a distance of about 10 mm from the base.

Unidirectional deflection cycles were applied with amplitude increasing by 5 mm from cycle-to-cycle. In general, testing with increasing deflection amplitude continued until and beyond the conventionally defined column ultimate deformation, which is taken where peak resistance in a cycle drops below 80% of the maximum recorded lateral resistance of the column. Often the conventionally defined ultimate deformation is associated with fracture of one or more vertical bars, or of the FRP wrapping.

A jack at the top of the column applied the axial load, acting against vertical rods connected to the laboratory strong floor through a hinge. The axial load was manually controlled and re-adjusted as the experiment progressed. With this setup, the P-Δ moment at the base of the column was equal to the axial load, times the ratio of the distance of the hinge from the column base to the point of application of lateral load (i.e., times $0.5/1.6=0.3125$). Bending moments reported in Tables 1–3 include the P-Δ contribution.

Tables 1–3 give the mean value of axial load during the test, normalized to the product of the external dimensions b and h of the full section and to f_c, as $\nu = N/bhf_c$. For the jacketed columns, the value of f_c for the jacket was used, as the flexural resistance and the deformation capacity of the jacketed member depended on the compressive strength of the compression zone, which normally does not extend beyond the thickness of the jacket.

LVDTs placed at opposite sides of the column in the direction of loading measured vertical displacements between the top of the footing and two column sections at 125 mm and 250 mm (50% or 100% of the depth of the original column section) from the base. These measurements provided the relative rotation and the mean axial strain within the corresponding length of the column above the base, including the effect of any pull-out of vertical bars from the footing.

3. Experimental Behavior and Discussion

Columns (6)–(9) in Table 1 and (7)–(10) in Tables 2 and 3, present the moment, the drift ratio (chord rotation), the curvature at the base at yielding of the column, M_y, θ_y, and φ_y, respectively, as well as the fixed-end rotation due to bar pullout. Columns (10)–(12) in Table 1 and (11)–(13) in Tables 2 and 3, give the ultimate drift ratio (chord rotation) and ultimate curvature at the base, θ_u and φ_u, respectively, as well as the fixed end rotation due to bar pull-out at ultimate. Yielding was identified with a distinct reduction in the slope of the moment-deflection curve and close to the corner point of a bilinear envelope that can be fitted to the ascending part of the force-deflection response, i.e., up to peak resistance. The drift ratio or chord rotation at the base of the column, θ, was the deflection at the point of lateral loading, divided by the shear span of 1.6 m. The curvature at the base of the column and the fixed-end rotation due to bar pullout were estimated from the rotations of the two instrumented sections assuming that the curvature is constant within the lowest 250 mm of the column. The reader is warned, though, that these values are subject to significant uncertainty, due to the assumptions involved in their estimation and noise in the rotation measurements.

The main features of the behavior and the mode of failure are noted at the last column of Tables 1–3.

3.1. Unretrofitted Columns

The five unretrofitted columns included one without laps (Q-0) and two columns for each lap-splice length (Q-0L1, Q-0L1a and Q-0L2, Q-0L2a). Specimens Q-0L1 and Q-0L2 had about the same axial load level as Q-0, while the effect of higher axial load was investigated in specimens Q-0L1a and Q-0L2a.

The force-displacement response of the five unretrofitted specimens is shown in Fig. 3. Figure 4 shows the damage of four of these columns after the end of the test. The behavior of all columns was controlled by flexure and exhibited a dramatic drop in resistance after ultimate strength. There was no clear effect of the lap length, as columns Q-0 without laps and Q-0L2 with the 25-bar diameter lap did not perform any better than Q-0L1 that had 15-bar diameter lapping and developed plastic hinging above the lap splice. Column Q-0L1a, by contrast, again with a 15-bar diameter lap, had lower ultimate deformation than Q-0L2a. The adverse effect of the level of the axial load on the post-peak strength degradation was much more evident than any effect of the lap-splicing. It is noteworthy that in specimens Q-0L1 (Fig. 4(b)), Q-0L1a and Q-0L2a (Fig. 4(d)) most of the final damage was above the lap splice.

3.2. RC-Jacketed Columns

The force-displacement response of the RC jacketed specimens is shown in Fig. 5. Not only the yield moment, but also the deformation capacity was enhanced by jacketing (by a

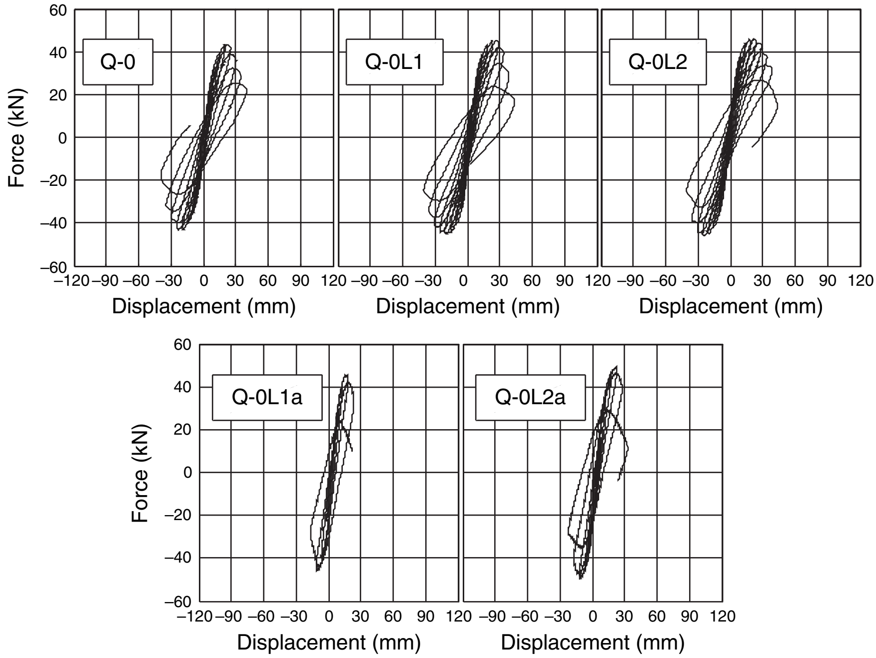
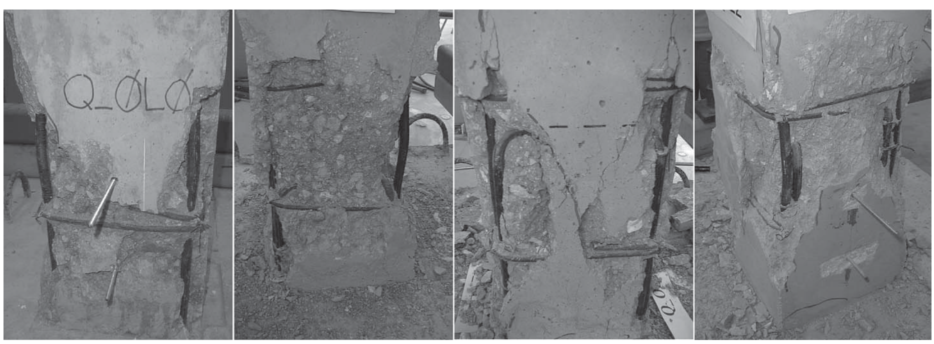


FIGURE 3 Effect of lap-splicing on unretrofitted columns; (top row): columns tested under lower axial load; (bottom row): columns tested under higher axial load.



(a) (b) (c) (d)

FIGURE 4 Damage of unretrofitted specimens after completion of test: (a) Q-0; (b) Q-0L1 with 15-bar diameter lapping; (c) Q-0L2 with 25-bar diameter lapping; (d) Q-0L2a with 25-bar diameter lapping.

factor of 3 at least, with respect to the unretrofitted columns). Even in specimens with lap-spliced bars in the original column, ultimate drift was around 5.5%. Peak resistance was almost independent of bar lapping in the original column inside the jacket. The effect of jacketing completely overshadowed that of the lapping and of its length. In columns with lap-spliced original reinforcement, only the post-ultimate strength drop in resistance seemed to accelerate.

As in the specimens without lapping, behavior was mainly governed by flexure, but this time with heavy bond-splitting and spalling all along the corner bars. The opening of

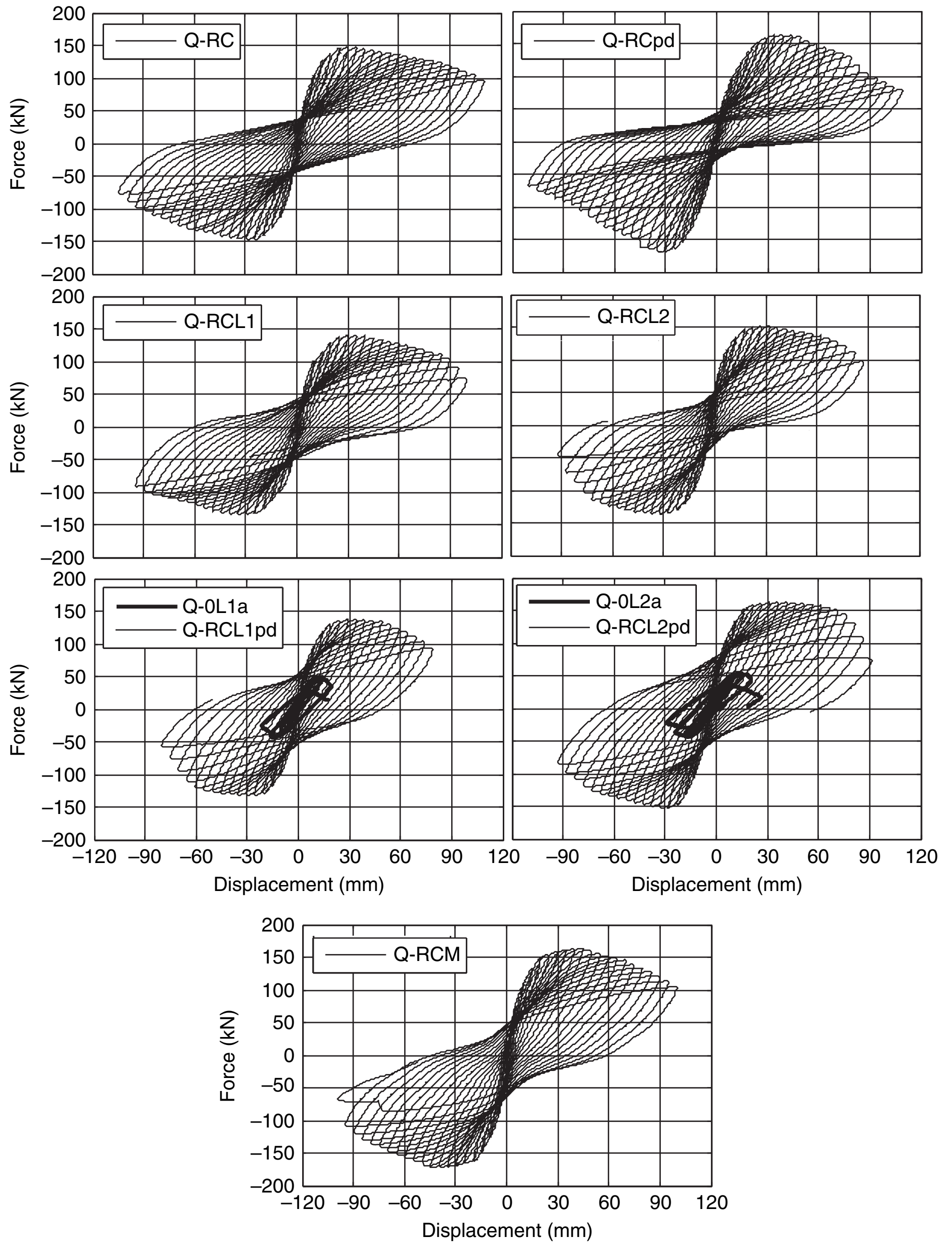


FIGURE 5 Effect of lap-splicing on RC jacketed columns: (top row) original column without lap splices, with or without previous damage; (2nd row) original column with lap splices, without previous damage; (3rd row) predamaged original column with lap splices; (bottom row) monolithic column.

any hairline diagonal cracks that may have developed up to maximum load kept decreasing after ultimate strength. Pre-damage of the original column (in specimen Q-RCpd beyond yielding and in Q-RCL1pd and Q-RCL2pd beyond ultimate deformation) did not seem to have a significant effect on the behavior of the subsequently jacketed specimens, Q-RCpd, Q-RCL1pd and Q-RCL2pd. This is depicted in the graphs at the third row of Fig. 5, where the response of specimens Q-RCL1pd and Q-RCL2pd (broken line) is compared to that of the old members Q-0L1a and Q-0L2a (continuous line). The lower strength of the cast-in-place jacket of Q-RCpd was a possible factor for the heavy bond splitting/spalling along its corner bars, without any apparent effect, though, on column lateral force and deformation capacity.

The damage of columns Q-RC, Q-RCL1pd, and Q-RCL2pd after the test is shown in Figs. 6(a)–(c).

The behavior and ultimate deformation of the monolithic column Q-RCM was very similar to that of the jacketed columns. The absence of the 4, 14 mm smooth bars, emulating the vertical reinforcement of the old column, did not have an apparent adverse effect on the yield or ultimate moment, or the stiffness of the column.

3.3. FRP-Wrapped Columns

Regardless of the number of layers of FRP material and the lap length, FRP-wrapped specimens behaved far better than their unretrofitted counterparts. In general, member deformation capacity increased by a factor of about 2.5, achieving a drift ratio at conventional failure (20% drop in resistance with respect to peak resistance) from 4.7% to above 7%, and invariably sustaining a maximum drift close to 7% by the end of the test.

As shown in Fig. 7 and Table 3, the increase in force capacity and the post-peak strength deterioration in the column without lap splices were about the same, irrespective of the layers of FRP and the length of its application (except for specimen Q_P4H1a, discussed later). Deformation capacity was slightly improved, if four layers of CFRP are used instead of two. Figures 6(e) and 6(f), show specimens Q-P4H2 and Q-P2H1, respectively, at the end of the test.

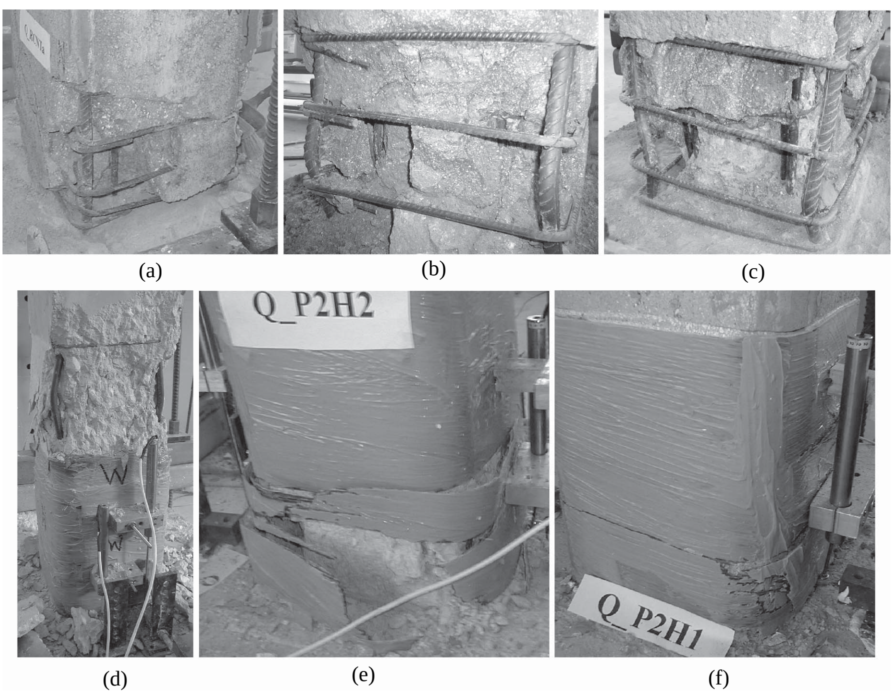


FIGURE 6 Damage of retrofitted columns: (a) Q-RC; (b) Q-RCL1pd; (c) Q-RCL2pd; (d) Q-P4H1a; (e) Q-P2H2; (f) Q-P2H1.

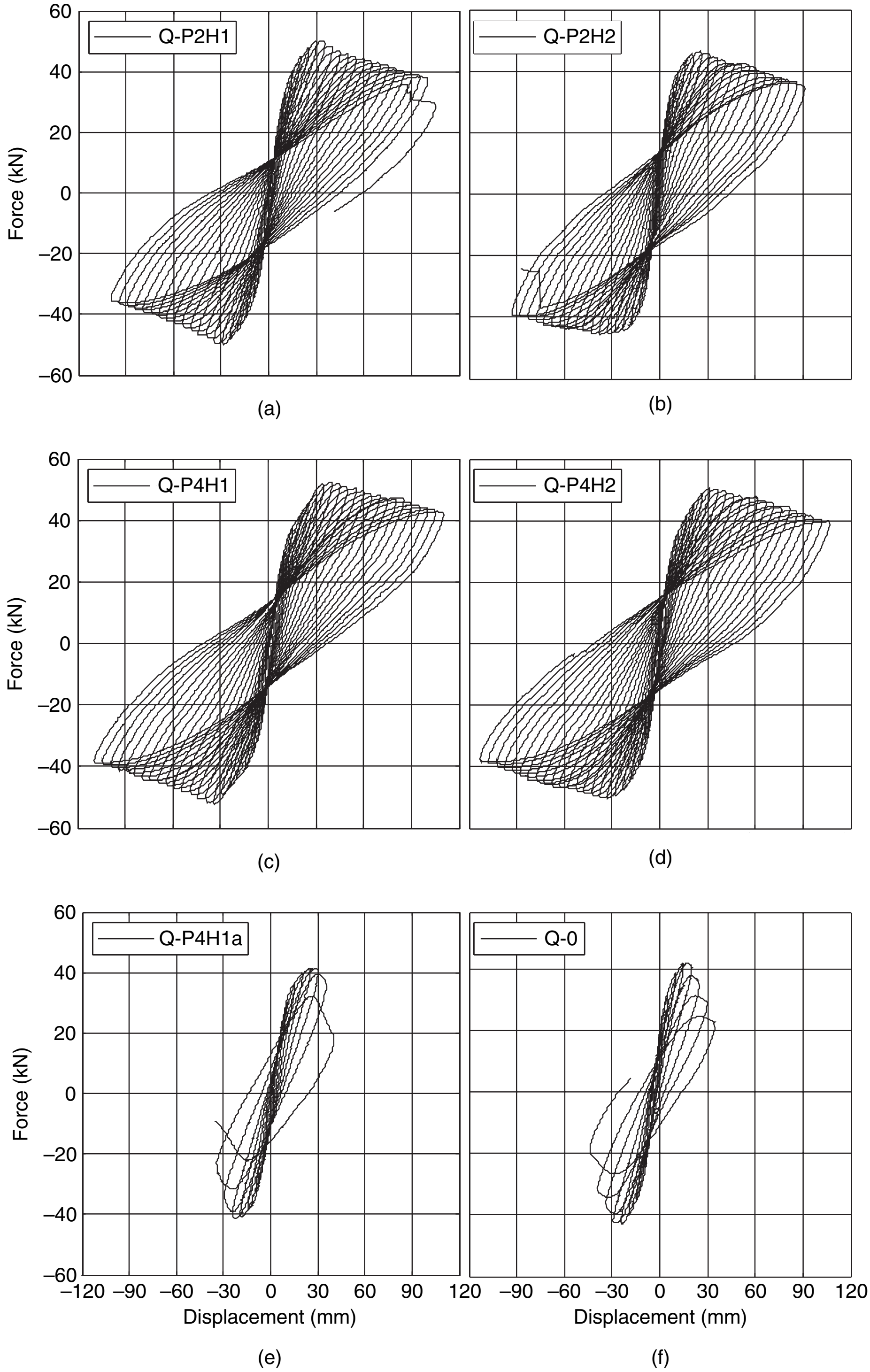


FIGURE 7 Force-deflection response of columns without lap splice: (a), (b) two CFRP layers over 0.3 m (a), over 0.6 m (b); (c)–(e) four CFRP layers over 0.3 m (c) and (e), over 0.6 m (d);(f) unretrofitted column.

Columns with either short lap length (L1 in Fig. 8) or long one (L2 in Fig. 8), wrapped with two or four layers of CFRP over the lower 0.6 m of the column, have also very satisfactory performance. Specimen Q-P2L1H2 sustained reversed deformation cycles up to 6% drift before failing by rupture of the FRP during the last cycle of applied displacement. The companion specimen with the same lap length but retrofitted with four layers of CFRP (Q-P4L1H2) had the same peak resistance, but a slower rate of strength degradation after the peak. No sign of rupture of the FRP jacket was observed before the end of the test at 6.8% drift ratio. When the FRP jacket was removed after the end of both tests, disintegration of the concrete inside the jacket was found.

The load-deformation pattern observed in columns with short lapping (i.e., Q-P2L1H2, Q-P4L1H2) did not change when the lap length was longer (Q-P2L2H2 and Q-P4L2H2), as demonstrated in Figs. 8(d) and (f), respectively: very stable behavior, with both specimens exhibiting the same force resistance and gradual strength degradation after peak force. In both specimens the FRP ruptured at the end of the test.

The effect of the length over which the FRP wrapping was applied (0.6 m vs. 0.3 m) was studied through the following specimens:

- columns Q-P4H1 and Q-P4H1a, in Figs. 7(c) and (e), respectively, both with four CFRP layers over 0.3 m and column Q-P4H2 in Fig. 7(d) with the same number of layers but over 0.6 m, all without lapping; and
- column Q-P4L1H1 with four CFRP layers over 0.3 m in Fig. 9(a), vs Q-P4L1H2 in Fig. 9(b) with the same number of layers over 0.6 m, both with 15-bar diameter lapping.

In column Q-P4L1H1 the CFRP layers over the lowest 0.3 m of the column covered 140% of the lap length.

The pair of columns Q-P4L1H1 in Fig. 9(a) and Q-P4L1H2 in Fig. 9(b), and that of columns Q-P4H1 in Fig. 7(c) and Q-P4H2 in Fig. 7(d), did not show a clear effect of the length of application of the FRP. In column Q-P4H1a, by contrast, which was in every respect similar to Q-P4H1, plastic hinging and subsequent column failure took place above the 0.3 m long FRP wrapping (Fig. 6(d)). As a result, the peak resistance, the post-peak strength decay and the ultimate deformation of this FRP-wrapped column were very similar to those of the unretrofitted column in Fig. 7(f). For a wrapping length of just 0.3 m, the bending moment (including the P-Δ effect) was about 20% larger at the base of the column than at the end of the wrapping. It seems, therefore, that the increase in flexural capacity at the base due to concrete confinement by the FRP and strain hardening of the reinforcement was at least 20%. So it prevented development of the full column flexural capacity at the base before yielding of the section just above the FRP wrapping and formation of a plastic hinge there. As a result, in the case of column Q-P4H1a, deformation capacity was controlled by the unconfined length above the FRP, instead of the well-confined region where yielding occured in the first place. It is therefore clear that the 0.3 m length of application of FRP cannot reliably cope with the post-yield increase in flexural capacity of the FRP-wrapped base of the column and prevent formation of a second plastic hinge and subsequent failure above the FRP wrapping.

Comparing the behavior of columns with short lapping (Figs. 8(a), (c), (e)), long lapping (Figs. 8(b), (d), (f)), or continuous bars (Figs. 7(b)-(e)), all wrapped with two or four CFRP jackets, it is concluded that, for smooth bars with 180° hoops at the ends and lapping of at least 15-bar diameters, lap length does not influence the force and cyclic deformation capacity and the rate of strength degradation in the rehabilitated column. Nonetheless, energy dissipation seems to be slightly affected by lap length, as evidenced by the reduction in the width of hysteresis loops observed when going from columns with

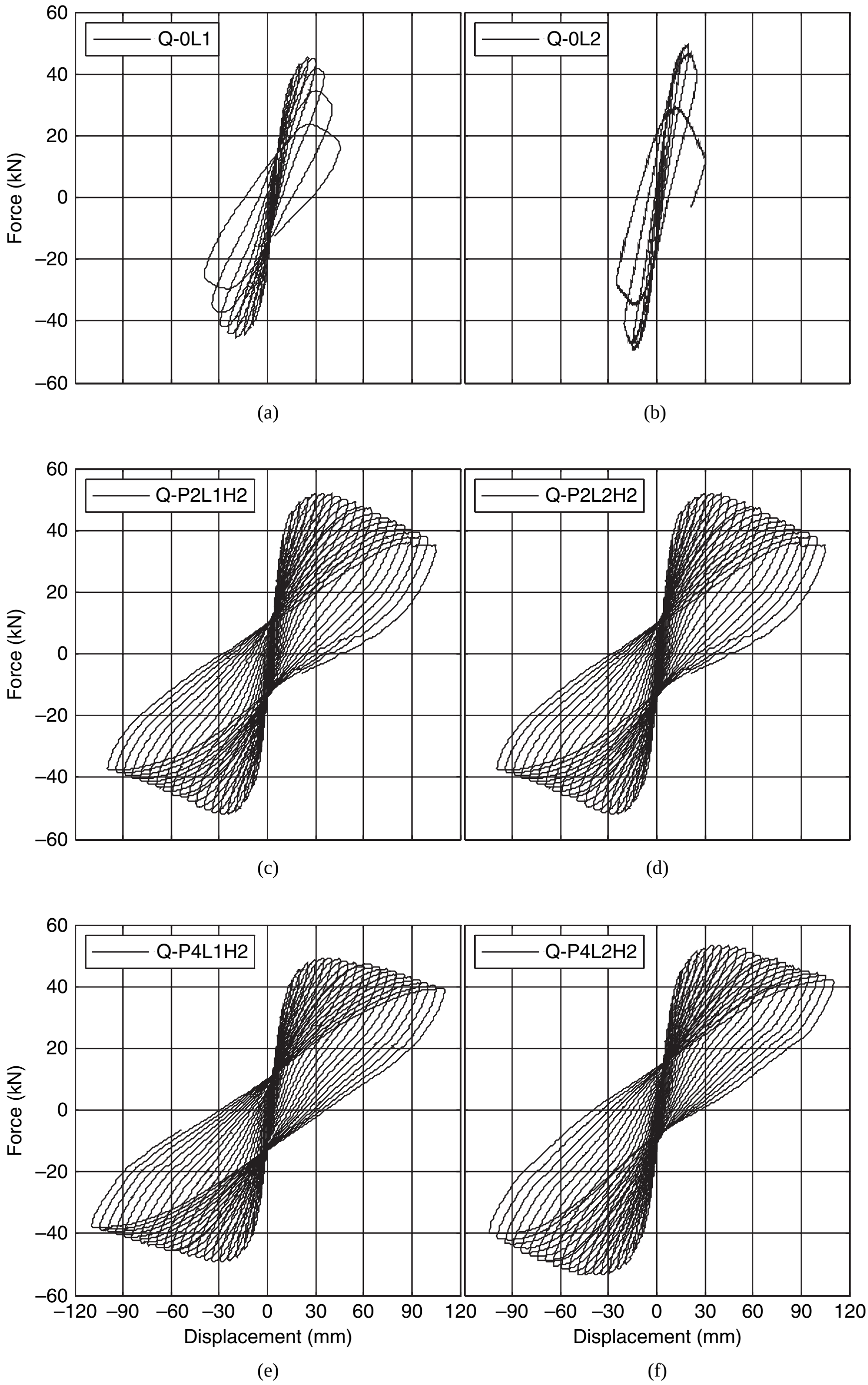


FIGURE 8 Force-deflection response of specimens with 15-bar diameter lapping (left column) or 25-bar diameter lapping (right column): (a), (b) unretrofitted specimens; (c), (d), specimens wrapped with two CFRP layers over 0.6 m; (e), (f): specimens wrapped with four CFRP layers over 0.6 m.

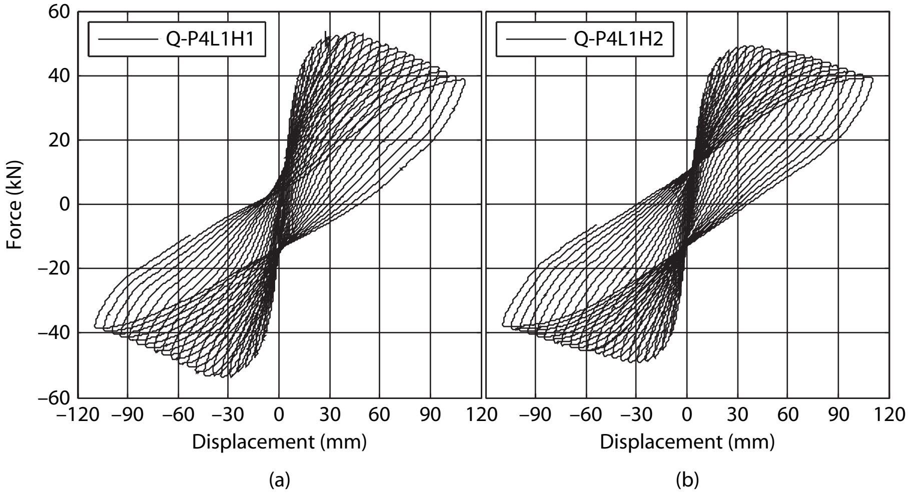


FIGURE 9 Columns with 15-bar diameter lapping and four FRP layers applied over: (a) 0.3 m; (b) 0.6 m.

continuous bars (Figs. 7(b)-(e)), to those with long lapping (Figs. 8(b), (d), (f)) and then to short lapping (Figs. 8(a), (c), (e)).

3.4. Mean Axial Strain at the Bottom of the Columns

The evolution of the mean axial strain at the center of the section over the lowest 250 mm above the base with the lateral deflection of the top of the column is presented in Fig. 10 for the unretrofitted and the RC-jacketed columns and in Fig. 11 for the FRP-wrapped ones. These strains were derived from the average of the LVDT measurements on opposite sides of the column. In RC-jacketed specimens, they essentially reflect the axial displacement of the lower-most 250mm of the jacket with respect to the footing. The change in column length in each cycle of loading (nearly) in proportion to the lateral deflection was due to the bending according to the plane-sections hypothesis.

The most interesting result in Figs. 10 and 11 is the ratcheting axial shortening or extension with the cyclic deflection. Columns with high normalized axial load $\nu = N/bhf_c$, such as the un-retrofitted specimens in the left column of Fig. 10 and all FRP-wrapped ones in Fig. 11 (all of which had ν between 0.4 and 0.45), exhibited ratcheting axial shortening, as load cycling led them beyond yielding and towards failure, due to accumulation of permanent compressive strains in the concrete (the diagram for column Q-L1 does not show ratcheting shortening, because the critical region turned out to be above the splice, i.e., above the region instrumented with LVDTs).

The large ratcheting axial elongation of the specimens in the central column of Fig. 10 (except the monolithic one at the top) was not so much due to their very low normalized axial load (owing to the very high compressive strength of their jacket shotcrete), but to the lack of any positive measures to connect the old concrete to the jacket.

In the predamaged specimens of the right column of Fig. 10, the shotcrete that replaced the spalled or damaged and removed concrete near the base of the old column improved the connection of the jacket. Moreover, in specimens Q-RCL1pd and Q-RCL2pd, which showed the least ratcheting axial elongation (or even axial shortening in Q-RCL2pd when failure was approaching) the full lateral surface of the old column had

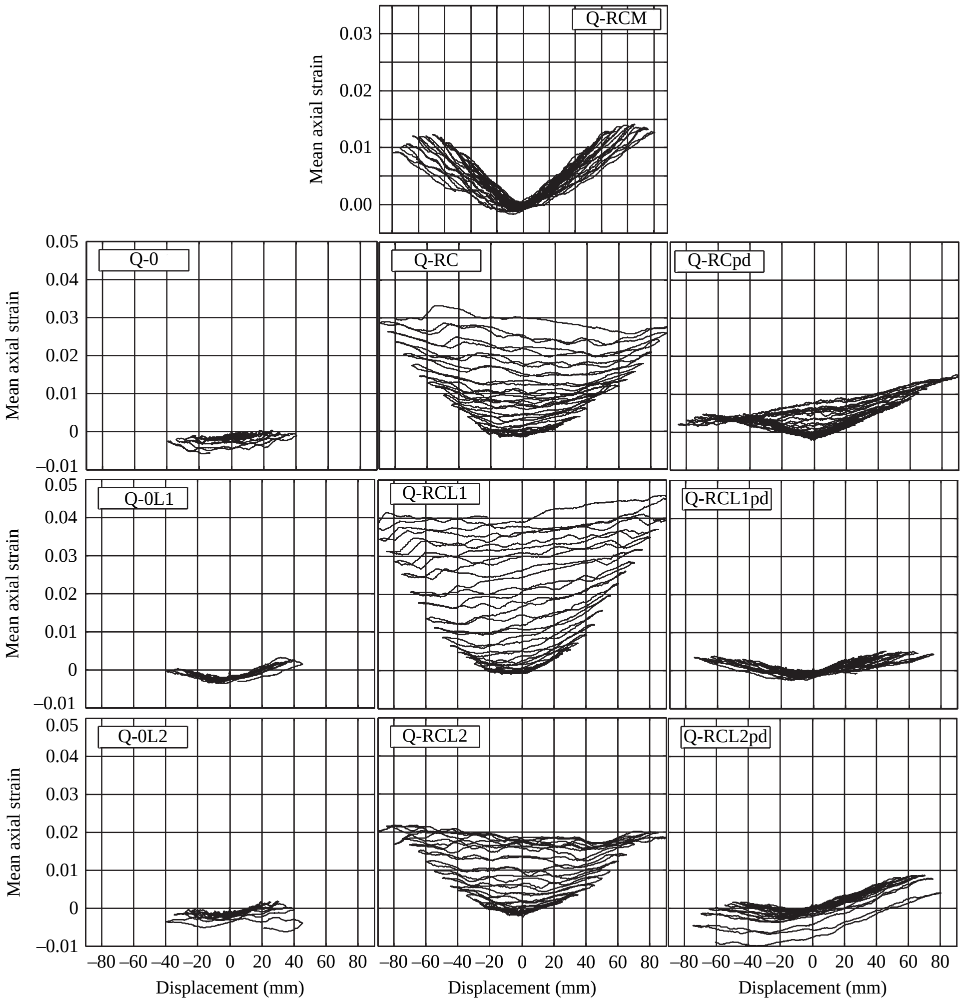


FIGURE 10 Evolution of mean vertical strain at center of section (over lower 250 mm from base) with lateral displacement of column: (top) monolithic column; (2nd row) original column without lap splices, with or without previous damage; (3rd row) original column with 15-bar diameter lapping; (bottom row) original column with 30-bar diameter lapping; (left column) unretrofittted specimen; (center) RC-jacketed columns; (right column) RC-jacketed columns with pre-damage and roughened interface.

been artificially roughened before jacketing. Note that, four columns had been tested by Bousias *et al*. [2006] after jacketing, with various positive measures to connect the old concrete to the jacket, but otherwise similar to Q-RC; they all showed ratcheting axial behaviour similar to that of Q-RCL1pd and Q-RCL2pd with the rough interface.

The large ratcheting elongation near the bottom of the columns which were jacketed without any positive measures to connect the old concrete to the jacket suggests significant slippage of the jacket with respect to the old column inside, which remained fixed to the footing during the test. There was no manifestation of any deleterious effects of this slippage on the global cyclic behaviour of the columns in Fig. 5 and on the parameters that summarize it in Table 2. The results by Bousias *et al*. [2006] provide further support to this conclusion.

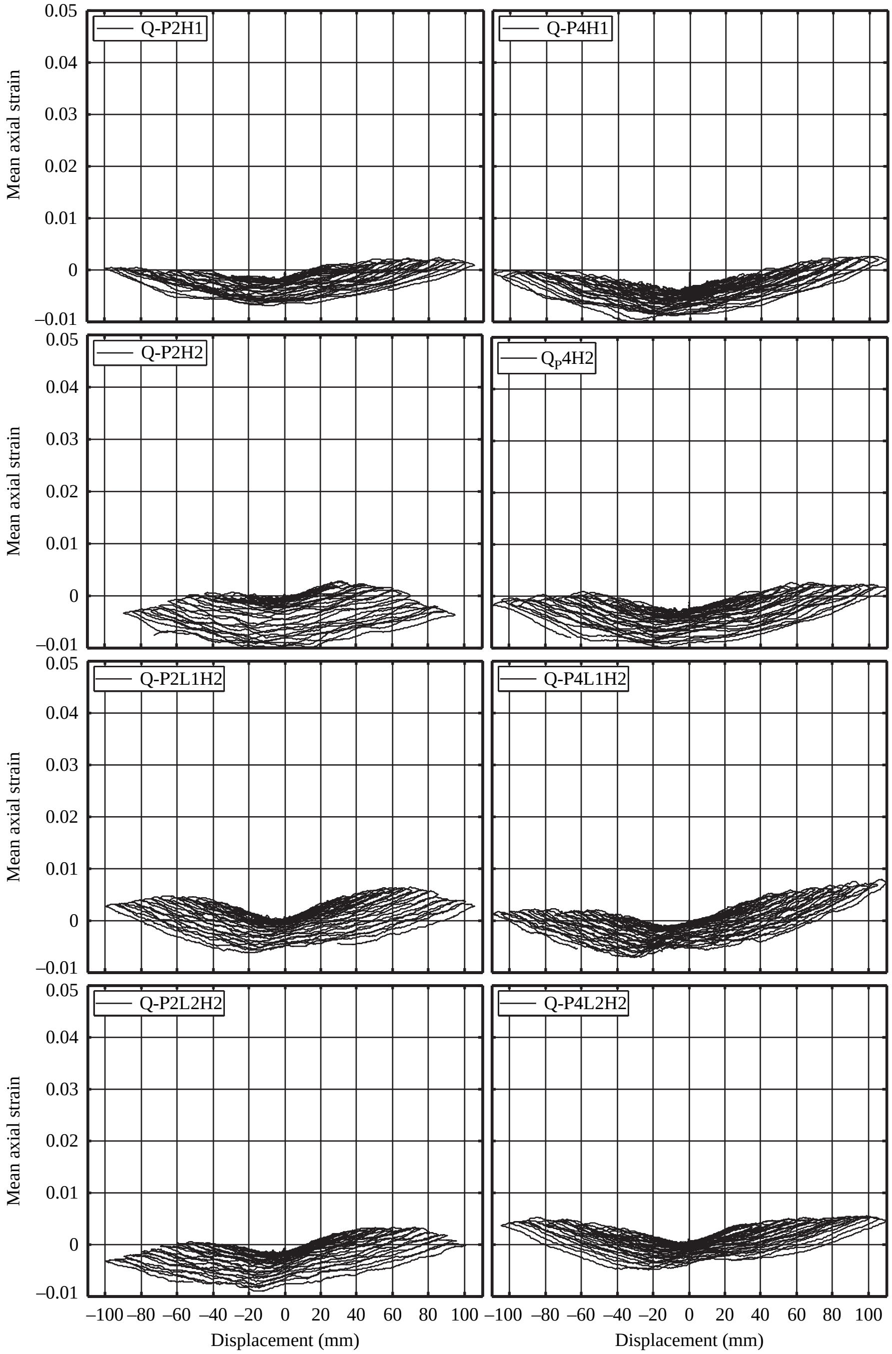


FIGURE 11 Evolution of mean vertical strain at center of section (over lower 250 mm from base) with lateral displacement of column: (top) column without lap splices and FRP plies over lower 0.3 m; (2^{nd} row) column without lap splices and FRP plies applied over 0.6 m; (3^{rd} row) column with 15-bar diameter lapping and FRP plies applied over 0.6 m; (4^{th} row) column with 25-bar diameter lapping and FRP plies applied over 0.6 m; (left column) specimens retrofitted with two FRP plies; (right column) specimens retrofitted with four FRP plies.

4. Conclusions

The conclusions of the present study are limited to rectangular columns with amount and detailing of reinforcement fairly similar to those studied in this article and for axial load below balanced load.

Old-type columns with smooth (plain) vertical bars have low deformation and energy dissipation capacity under cyclic loading, which is, however, not impaired much further by lap splicing of the vertical bars at floor level, provided that the lapping is at least 15 bar-diameters. A lap length of at least 15-bar diameters seems to sufficiently supplement the 180° hooks for the transfer of forces. RC jacketing of such columns increases their deformation capacity to levels sufficient for earthquake resistance, irrespective of the presence and length of lap splicing. Previous damage of the old column by cyclic loading beyond its peak resistance does not reduce noticeably the effectiveness of the jacket. Lack of positive measures to connect the old concrete to the jacket causes significant slippage at their interface, but does not adversely affect the lateral load resistance, deformation capacity or energy dissipation of the jacketed column.

In FRP-wrapped columns with smooth bars and hooked ends, the number of layers above two, or even the presence and length of lapping, were not found to have a systematic effect on the resistance and cyclic deformation capacity and the rate of strength degradation. However, a decrease in lap length seemed to reduce energy dissipation in the FRP-retrofitted columns. Overall, FRP wrapping of just the plastic hinge and any splice region is more effective than concrete jacketing in enhancing the deformation and energy dissipation capacity of old-type columns having smooth bars with or without lap-splicing. It is notable that FRP-wrapping of an end portion of the member length equal to just 20% of the shear span is not sure to preclude plastic hinging and early member failure outside the FRP-wrapped length of the column.

Acknowledgments

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