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Effectiveness of R/C jacketing of substandard R/C columns with short lap splices

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Abstract. The effectiveness of a retrofitting method for concrete columns with particular weaknesses is experimentally evaluated and presented in this paper. Structural deficiencies namely the inadequacy of transverse reinforcement and short length of lap splices are very common in columns found in structures built prior to the 1960s and 1970s. Recent earthquakes worldwide have caused severe damages and collapses of these structures. Nevertheless, the importance of improving the load transfer capacity between the deficiently lap-spliced bars is usually underestimated during the strengthening procedures applied in old buildings, though critical for the safety of the residents' lives. Thus, the seismic performance of the enhanced columns is frequently overestimated. The retrofitting approach presented herein involves reinforced concrete jacketing of the column sub-assemblages and welding of the lap-spliced bars to prevent the splice failure and conform to the provisions of modern design Codes. The cyclic lateral loading response of poorly confined original column specimens with insufficient lap splices and the seismic behavior of the retrofitted columns are compared. Test results clearly demonstrate that the retrofitting procedure followed is an effective way of significantly improving the seismic performance of substandard columns found in old buildings.

Keywords: reinforced concrete; columns; R/C jackets; welding; lap splice; reversed cycling loading; seismic retrofitting

1. Introduction

One of the most significant weaknesses of reinforced concrete buildings, designed and constructed prior to the 1960s and 1970s, is the short length of lap splices in columns. Mainly due to lack of capacity design approach and poor detailing of reinforcement, low-confined lap splices of insufficient length, located at the lower part of the columns just above the floor slab are very common in these buildings worldwide. The lap splices were designed only as compressive splices, with typically short lengths of 20 or 24 times (or similar) the diameter of the longitudinal column reinforcement. As a result the substantial tensile forces developing on the bars during earthquakes cannot be transferred between the spliced bars (Cho and Pincheira 2006, Melek and Wallace 2004, Lynn *et al.* 1996, Valluvan *et al.* 1993, Chai *et al.* 1991). Moreover, the lap-splice length requirements for tension are significantly higher than those for compression. Without the beneficial influence of confinement, slipping of the spliced bars occurs before the development of

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the column nominal moment capacity, resulting in premature degradation of the column strength, stiffness and energy dissipation capacity. Under large displacement reversals the excessive dilation of splitting cracks along the (poorly confined) deficiently lap-spliced bars causes early loss of the column cover concrete and eventually rapid deterioration of the column strength, due to the splice failure (Priestley *et al.* 1996). Bond deterioration and slipping of the inadequately lap-spliced bars dominate the load-deflection response of columns representative of those found in old buildings. Hence, the retrofitting techniques applied to old buildings should improve the load transfer capacity between the deficiently lap-spliced bars and ensure the yielding of the longitudinal column reinforcement.

Adding external confinement to the poorly detailed old columns has proved to increase the bond strength between the steel bars and concrete. Harajli (2009) used external FRP jackets to improve the bond strength of lap-spliced column reinforcement bars. A tri-uniform bond stress distribution model was proposed by Thai and Pimanmas (2011) for the prediction of the lap splice strength in columns retrofitted by FRP sheets. El Gawady *et al.* (2010) investigated the seismic behavior of reinforced concrete columns retrofitted by using CFRP jackets and conventional steel jackets. The bond behavior between steel bars and concrete, when steel wrapping jackets were used as means of external confinement, was studied by Choi *et al.* (2013). Karayannis and Sirkelis (2008) combined epoxy resin injection and CFRP jacketing for the repair and strengthening of exterior beam-column joints. Reinforced concrete jacketing is also one of the earliest and most common retrofitting techniques used for the enhancement of substandard columns found in existing buildings (Engideniz *et al.* 2005, Melek and Wallace 2004). Rodriguez and Park (1994) experimentally investigated the effectiveness of reinforced concrete jackets in the repair and strengthening of substandard columns designed prior to the 1970s and noted substantial improvement of the overall seismic performance of the strengthened specimens with respect to the behavior of the as-built columns. Gomes and Appleton (1998) used reinforced concrete jackets for the retrofitting of seriously damaged columns under cyclic lateral loading and examined the influence of parameters such as the length of the jacket, the confinement offered by the jacket, the axial load of the column and the concrete strength of the jacket. In the experimental study of Julio and Fernando (2008) the influence of interface treatment on the seismic performance of columns enhanced with reinforced concrete jackets and subjected to cyclic lateral loading was investigated. Karayannis *et al*. (2008) examined the effectiveness of local retrofitting exterior beam-to-column joints with thin reinforced concrete jackets. Ten original specimens were constructed and imposed to increasing cyclic lateral loading. After the seismic loading, thin reinforced concrete jackets with reinforcement consisting of small diameter bars were locally constructed to encase the joint region and part of the columns and the beam, in order to improve the strength and energy dissipation capacity of the sub-assemblages, without causing significant changes in the initial size of the elements. The lateral performance of four exterior beam-to-column sub-assemblages with little amount or with lack of transverse reinforcement in the joint region strengthened by three-sided reinforced concrete jackets was studied by Tsonos (2001). One more specimen with lack of joint ties was repaired by a two-sided reinforced concrete jacket and subjected to cyclic lateral loading (Tsonos 2002). In both the cases of three-sided and two-sided jacketing, the strengthened specimens exhibited a desirable ductile failure mode with the formation of the plastic hinges in the beams, while simultaneously buckling of the beam bars was observed. Tsonos (2008) also investigated experimentally the performance of the reinforced concrete jacket system and of the high-strength fiber jacket system in the cases of post-earthquake and pre-earthquake retrofitting of columns and beam-to-column joints. Both repair and strengthening techniques found to be dkeffective. In particular, the reinforced concrete jacket system found to be more effective in a post-earthquake retrofitting of columns and of beam-to-column joints than the high-strength fiber jacket, while the two systems were equally effective in the case of pre-earthquake retrofitting. In another experimental study, Tsonos (2010) examined the effectiveness of two-sided and four-sided shotcrete and cast-in-place concrete jackets for the cases of pre-earthquake and post-earthquake retrofitting of columns and beam-to-column joints. The results obtained from the seismic loading of the specimens indicated that all the examined types of concrete jackets were equally satisfactory in the strengthening of existing old structures.

Despite the significant confinement provided by jacketing, bond slip deformations between concrete and the lap-spliced bars may continue to exist unless additional measures, aimed to improve the load transfer between reinforcement steel, are undertaken. Otherwise, the flexural strength and lateral performance of the pre-earthquake retrofitted columns are overestimated. In the present study the welding of the splices is combined with reinforced concrete jacketing to decrease the bar slipping and improve the overall seismic behavior of the enhanced columns.

2. Experimental program

The experimental research relating to the effectiveness of reinforced concrete jacketing of columns with substandard detailing of reinforcement and primarily with short lap splices is rather poor. In this paper the efficiency of a retrofitting method for columns representative of those found in structures built prior to the 1960s and 1970s was experimentally investigated. Eight cantilever column sub-assemblages of approximately 1:1.5 scale were designed and constructed. Four of them were original column specimens typical of existing old buildings, designed according to the low-standard requirements of old seismic Codes. These columns possessed the properties of substandard concrete columns with plain steel longitudinal and transverse reinforcement (S220) and normal weight concrete with low compression strength (C8/10), measured on 150x300 mm cylinders (Table 1). The specimens, named O_1, L_1, O_2 and L_2, had inadequate shear reinforcement and insufficient lap splices of length equal to 20 and 24 times the bar diameter, located inside the critical region of the columns. A fifth original sub-assemblage, C_2, was constructed with similar reinforcement details but with continuous longitudinal steel bars. Reinforcement details and material properties of the original specimens are shown in Tables 1-2 and in Fig. 1. The original specimens O_1 and O_2 were subjected to cyclic lateral loading without been retrofitted, while sub-assemblages L_1, L_2 and C_2, were pre-earthquake strengthened by reinforced concrete jacketing. For the specimens with short lap splices of $20d_b$ and $24d_b$, L_1 and L_2 respectively, additional measures to the confinement provided by jacketing were undertaken, aimed at successfully creating the continuity of the longitudinal column reinforcement and preventing bond-slip deformations during earthquake motions. Thus, jacketing was combined with the welding of the splices along the splice height. Consequently, the retrofitting of columns L_1 and L_2 provided well-confinement and significant improvement of the bond between steel bars and concrete to prevent the premature splice failure and ensure the development of the longitudinal steel yielding stress. After the interventions columns L_1 and L_2 were designated RWL_1 and RWL_2, respectively. The enhanced column RC_2 was representative of the optimum load transfer conditions with respect to columns with lap splices and was used as the control specimen. The three pre-earthquake retrofitted sub-assemblages RWL_1, RWL_2, RC_2 and the two original columns, O_1, O_2, were subjected to inelastic cyclic lateral deformations to represent the equivalent of severe earthquake motions and test results revealed that the columns with the deficient lap splices achieved a significant improvement of their overall seismic performance when strengthened as described previously.

Table 1 Concrete compression strength and steel yield stress of the original specimens

Original Specimen	C_2	O_1	O_2
Concrete compression strength (MPa)	8.94	9.81	8.80
Steel yield stress (MPa) Longitudinal bars / ties	374/263.5	374/263.5	374/263.5

(a) the cover concrete of the original specimens is chipped away

(b) welding of lap splices in the original column and R/C jacket reinforcement

(c) insertion of epoxy resign

Fig. 1 Retrofitting of the column specimens with deficient lap splices

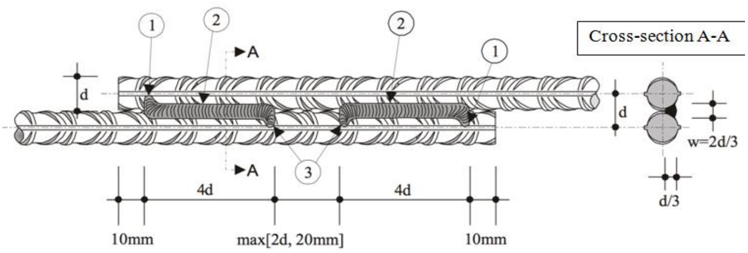


Fig. 2 Welding of the lap spliced bars according to the C.S.R.T.C (2008). 1: welding starts 10 mm from the bar end, 2: welding direction, 3: space between the welds

Table 2 Details of the original and retrofitted specimens

	Original columns			Retrofitted columns		
Specimen	C_2	O_1*, L_1	O_2*, L_2	RC_2	RWL_1	RWL_2
Lap splice length (mm)	–	200	240	–	200 (original column)	240 (original column)
Column length (mm)	980	980	980	980	980	980
Dimensions of column section (mm)	200x200	200x200	200x200	300x300	300x300	300x300
Longitudinal reinforcement ratio	0.0079	0.0079	0.0079	0.007	0.007	0.007
Longitudinal reinforcement of the original columns	4Ø 10 S220 cont.	4 Ø 10 S220 lap-spliced	4 Ø 10 S220 lap-spliced	4 Ø 10 S220 cont.	4 Ø 10 S220 lap-spliced	4 Ø 10 S220 lap-spliced
Longitudinal reinforcement of the R/C jacket	–	–	–	4 Ø 10 B500C	4 Ø 10 B500C	4 Ø 10 B500C
Transverse reinforcement of the original columns	Ø 6/200 S220	Ø 6/200 S220	Ø 6/200 S220	Ø 6/200 S220	Ø 6/200 S220	Ø 6/200 S220
Transverse reinforcement of the R/C jacket (along the critical height)	–	–	–	Ø 8/80 B500C	Ø 8/80 B500C	Ø 8/80 B500C
Welded lap splices	–	–	–	–	welded	welded
Loading pattern	Cyclic lateral loading with constant axial load					

* Specimens O_1 and O_2 are tested without been retrofitted

Table 3 Nominal moment capacity of the retrofitted columns

$\boldsymbol{f'_{ck}}$*(MPa)	$\boldsymbol{N_{sd}}$**(kN)	$\boldsymbol{M_R}$(kNm)
36.88	150	57.75

*$\boldsymbol{f'_{ck}}$: Calculated by (Eq.(1)); **$\boldsymbol{N_{sd}}$: Constant axial compressive load

3. Retrofitting process

The enhanced columns RWL_1, RWL_2 and RC_2 conformed to the modern code standards for seismic design. During the pre-earthquake strengthening process the cover concrete of columns L_1, L_2 and C_2 was chipped away (see Fig. 1(a)). Before the construction of the reinforced concrete jackets additional measures were undertaken to ensure the increasing of load transfer and the continuity of the spliced bars. Hence, the spliced bars of columns RWL_1 and RWL_2 were welded-up as shown in Fig. 2, to satisfy the requirements of the Code of Steel Reinforcement Technology for Concrete (CSRTC 2008). A four-sided cement grout jacket with additional longitudinal and transverse reinforcement was subsequently constructed according to the recommendations of the Greek Code for the Design of Reinforced Concrete Structures (C.D.C.S., 2000) and to the provisions of Eurocode 2 and 8 (2004). The retrofitted columns are shown in Fig. 4(b). Reinforcement details of the strengthened specimens are presented in Table 2. The longitudinal jacket reinforcement bars were anchored inside the foundation block of the sub-assemblages, after drilling the needed holes and cleaning them with air-pressure. A flowable, nonshrink epoxy resin was inserted inside the holes by syringe to achieve the bonding between the new bars and the concrete (see Fig. 1(c)). Connection between the longitudinal reinforcement of the jackets and the bars of the original columns was achieved by s-shaped steel segments which were welded to the bars (see Figs. 1(b) and 4(b)), according to the provisions of the Code of Steel Reinforcement Technology for Concrete (CSRTC 2008). The transverse reinforcement of the reinforced concrete jackets consisted of B500C 8mm diameter deformed closed ties with welded ends (welded according to CSRTC (2008)). A premixed, non-shrink, rheoplastic, flowable, and non-segregating mortar of high strength was used for constructing the cement grout jacket of the specimens.

4. Shear design of the R/C jackets

The reinforced concrete jackets of the enhanced column specimens were designed in shear according to the provisions of the Greek Code for the Design of Concrete Structures (C.D.C.S., 2000) and the provisions of Eurocode 2 and 8. The concrete compression strength of the retrofitted columns is given by Eq. (1).

The values of shear design parameters of the retrofitted sub-assemblages are shown in Tables 3-4. According to the provisions of C.D.C.S. 2000 the shear strength of the enhanced column sub-assemblages, V_{Rd3}, is the sum of the shear force resisted by concrete in the compressive zone, V_{cd}, and of that resisted by the transverse reinforcement of the columns, V_{wd}. Moreover, the value of V_{Rd3} must exceed the maximum shear force, V_{sd}, which corresponds to the nominal moment capacity of the specimens, Eq. (2). Eqs. (3) and (4) give the values of forces V_{cd} and V_{wd}, respectively. The shear strength of the concrete compression strut, V_{Rd2}, is given by Eq. (5) and must also exceed the value of V_{sd}. Eventually, closed B500C ties of 8 mm diameter spaced at 80 mm should be placed along the critical height of the columns according to the C.D.C.S 2000.

$$\frac{A_{s,ex} \cdot f_{ck,ex} + A_{s,j} \cdot f_{ck,j}}{A_{s,ex} + A_{s,j}} = f_{ck}' \tag{1}$$

$$V_{Rd3} = V_{wd} + V_{cd} > V_{sd} \tag{2}$$

$$V_{cd} = 0.3 \cdot V_{Rd1} \quad \text{for} \quad v > -0.1$$

where $$V_{Rd1} = \left[\tau_{Rd} \cdot k \cdot (1.2 + 40 \cdot \rho_l) + 0.15 \cdot \sigma_{cp}\right] \cdot b_w \cdot d \tag{3}$$

$$V_{wd} = \left(\frac{A_{sw}}{s}\right) \cdot 0.9 \cdot d \cdot f_{ywd} \tag{4}$$

$$V_{Rd2} = \frac{1}{2} \cdot v \cdot f'_{cd} \cdot b_w \cdot z > V_{sd} \tag{5}$$

In Eqs. (1)-(5) $A_{s,ex}$, $A_{s,j}$, $f_{ck,ex}$ and $f_{ck,j}$ are the cross section area and the characteristic concrete strength of the existing column and of the R/C jacket, respectively, τ_{Rd} is a design factor of shear strength where $\tau_{Rd} = 0.25 \cdot f_{ctk0.05}/1.5$ and $f_{ctk0.05} = 0.7 \cdot f_{ctm}$ where $f_{ctm} = 0.3 \cdot f_{ck}^{2/3}$, k is equal to $1.6 - d$ where d is the effective depth of the cross-section, $\rho_l = A_{sl}/(b_w \cdot d) \leq 0.02$ where A_{sl} denoted the cross section area of the longitudinal reinforcement under tension, $\sigma_{cp} = N_{sd}/A_c$ where N_{sd} is the axial force applied to the column and A_c the cross section area of the retrofitted column, A_{sw} is the cross section area of the transverse reinforcement, s is spacing of the stirrups, f_{ywd} is the yield strength of the transverse reinforcement, v is the normalized design axial force, b_w the cross-sectional depth of the column and $z = 0.9 \cdot d$.

The strengthened columns were also designed to conform to the provisions of Eurocode 2 and 8. Therefore, the control of the concrete compression strut adequacy was made according to equation Eq. (6). In Tables 4 and 5 the Eurocode provisions for the shear reinforcement are presented. The transverse reinforcement demand is finally given by Eq. (7), where it is recommended that $V_{Rd,s} \geq V_{sd}$. Ultimately, in order to conform to the provisions of the Eurocode for the shear design of reinforced concrete columns, closed B500C ties of 8 mm diameter spaced at 80 mm should be placed along the critical height of the strengthened sub-assemblages.

$$V_{Rd,max} = a_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}/(\cot\theta + \tan\theta) \tag{6}$$

$$V_{Rd,s} = \left(\frac{A_{sw}}{s}\right) \cdot z \cdot f_{ywd} \cdot \cot\theta \geq V_{sd} \tag{7}$$

In Eqs. (6) and (7) a_{cw} is a coefficient taking account of the state of the stress in the compression strut, v_1 is the strength reduction factor for concrete cracked in shear with recommended value of $1 - f_{ck}\ (MPa)/250$ and $\theta = 21.8°$.

5. Test setup and loading history

The seismic tests of the original and of the pre-earthquake strengthened sub-assemblages were conducted in the test frame shown in Fig. 3. The structures were fixed to the test frame with post-tensioned bars (bolts) and thus, the horizontal and vertical displacement and the rotation of the foundation block of each column were restrained. All specimens were subjected to a large number of inelastic cycles applied by slowly displacing the column's free end. The lateral load was applied to the free end of each column by a two-way actuator and the applied loads were measured by using a load-cell. An axial load of 150 kN was also imposed by a hydraulic jack (see Fig. 3) placed on top of the columns, perpendicular to the lateral loading direction and was controlled to keep constant during the seismic tests. The load point displacement was measured by a calibrated linear variable differential transducer (L.V.D.T.). Strain gages were installed in various positions of the longitudinal reinforcement of each specimen. All the enhanced sub-assemblages were loaded transversely according to the load history shown in Fig. 5. The top displacement amplitudes correspond to drift ratios that are shown in Table 6.

Table 4 Shear design of R/C jackets according to the provisions of C.D.C.S. 2000 and Eurocode 2 and 8

Design of the R/C Jacket in Shear										
	C.D.C.S. (2000)							Eurocode 2, 8		
V_{sd}* (kN)	V_{Rd1} (kN)	V_{cd} (kN)	V_{Rd2} (kN)	V_{wd} (kN)	V_{Rd3} (kN)	Ties along critical height		$V_{RD,max}$ (kN)	$V_{Rd,s} \geq V_{sd}$ (kN)	Ties along critical height
58.93	73.93	22.18	462.43 >V_{sd}	132.07	154.25 >V_{sd}	∅8/80 mm B500C		526.59 >V_{sd}	For $V_{Rd,s} \geq V_{sd}$ s**≤450 mm	∅8/80 mm B500C

*V_{sd}: Shear force corresponding to the moment resistance of the retrofitted columns; **s: Spacing of the stirrups

Table 5 Column critical height and spacing of the transverse reinforcement according to Eurocode 2 and 8

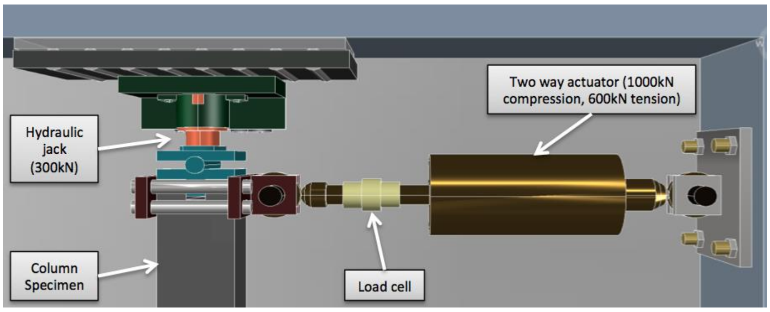
Eurocode 2 and 8		
	min∅$_w$: 8 mm	
Column critical height (m) EC 8 – 5.4.3.2.2.(4)	$s_{cl,t,max}$ (mm) Spacing of the ties along the critical height	$s_{cl,t,max}$ (mm) Spacing of the ties outside of the critical height
$max\left\{h_c, \frac{l_{cl}}{6}, 0.45\ m\right\}$	$min\left\{8 \cdot d_{bL,min}, \frac{b_o *}{2}, 175\ mm\right\}$	$min\{20 \cdot d_{bL,min} **, b_c, 400\ mm\}$
0.50	80	200

*b_o: width of the confined core (to the centerline of the hoops) $b_o = b - 2 \cdot (c + \emptyset_w/2)$, where $\emptyset_w$ denotes the diameter of ties and c is the width of the cover concrete, **$d_{bL,min}$: minimum diameter of the longitudinal steel bars

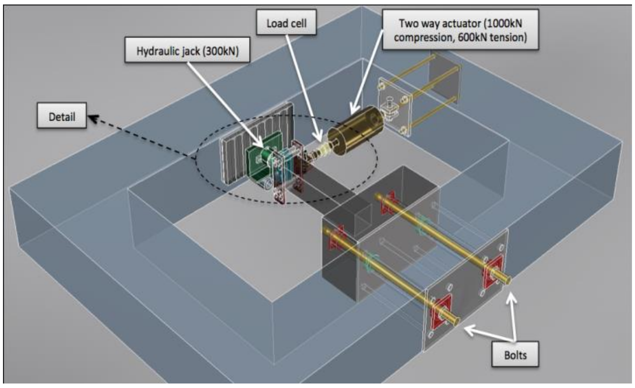
Table 6 Top displacement amplitude and inter-storey drift ratio

Top displacement amplitude ΔL and inter-storey drift angle ratio R											
Cycle of loading	1	2	3	4	5	6	7	8	9	10	11
ΔL (mm)	15	20	25	30	35	40	45	50	55	60	65
Drift: (ΔL/L*) %	1.53	2.04	2.55	3.06	3.57	4.08	4.59	5.10	5.61	6.12	6.63

*L: Column length



(a) Detail of the column free end - load point



(b) Test setup

Fig. 3 Test setup and detail of the column top where lateral and axial loads are imposed

6. Comparison of the test results

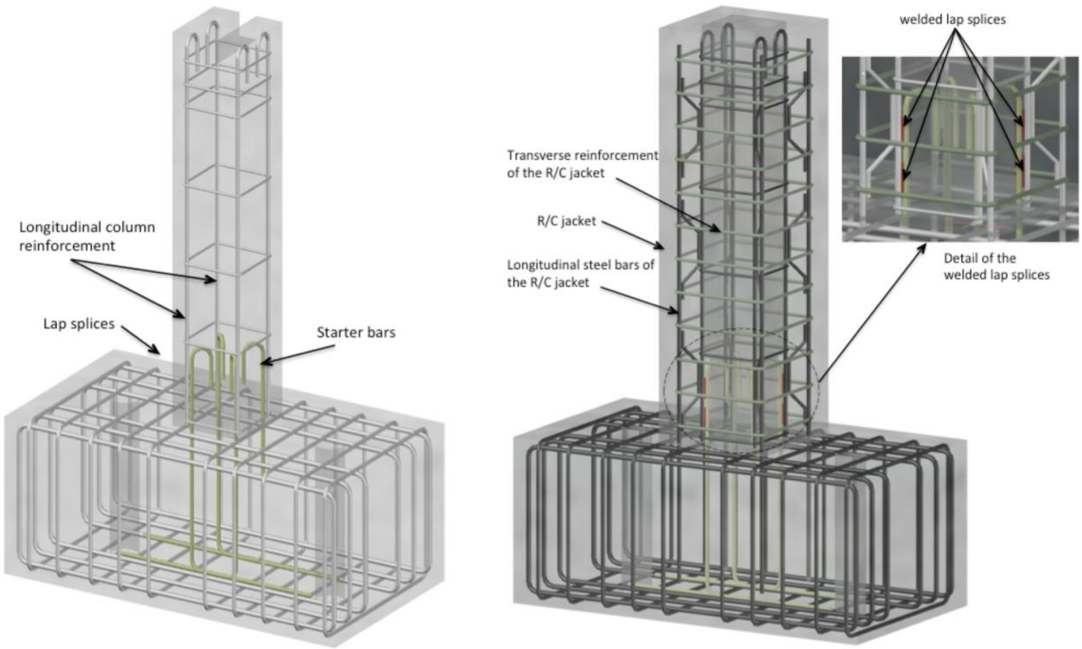
The effectiveness of the retrofitting technique applied to columns RWL_1 and RWL_2 in improving their overall response under large displacement reversals imposed by earthquakes is subsequently evaluated with respect to the cyclic lateral performance of the original columns O_1 and O_2 and that of the control specimen RC_2. Thus, the energy dissipation capacity, peak-to-peak stiffness and lateral strength of the sub-assemblages are compared in each cycle of the seismic loading.

In Fig. 6 the plots of applied shear-versus-displacement of the original and strengthened specimens are illustrated. The spindle-shaped hysteresis loops of the retrofitted columns RWL_1 and RWL_2 demonstrate the substantial improvement achieved, concerning energy dissipation capacity, with respect to the substandard column specimens O_1 and O_2. High values of energy were dissipated in the plastic hinges of RWL_1 and RWL_2 (see Fig. 7). These values exceeded in some cases even those of the control specimen RC_2. In the first cycle of lateral loading the energy dissipated of specimen RWL_1 was 69.62 percent higher than that of column O_1, while in the case of specimens RWL_2 and O_2 the corresponding value was 45.39 percent (see Figs. 7(a) and 7(b)).

During the incremental displacement of lateral loading the strengthened specimens RWL_1 and RWL_2 showed a significant gradual increase of energy dissipation, while in the case of the original columns energy dissipation capacity remained low until the seventh cycle (drift angle R equal to 4.59 percent). For lateral displacement of 45 mm, values of dissipated energy ratios RWL_1/O_1 and RWL_2/O_2 were equal to 5.2 and 5.4, respectively. Even for large displacements amplitude cycles of inter-storey drift, R, beyond 4.08 percent and up to the end of the seismic loading the retrofitted sub-assemblage RWL_1 continued to dissipate high values of seismic energy. After 11 cycles of lateral loading the dissipated energy ratio RWL_1/RC_2 was equal to 0.786, while values of ratio RWL_2/RC_2 were between 0.719 (in the first cycle) and over 1.00 for large displacements of inter-storey drift between 3.57 and 6.12 percent.

The peak-to-peak stiffness of the original (O_1 and O_2) and strengthened (RWL_1, RWL_2 and RC_2) column sub-assemblages is summarized in Fig. 8. A substantial increase in stiffness of the retrofitted columns RWL_1 and RWL_2, with respect to that of the original specimens O_1 and O_2, was observed. For lateral displacement of 15mm stiffness of column $RWL_1$was 2.62 times higher than the stiffness of the original sub-assemblage, L_1, while in the case of columns RWL_2 and L_2 the stiffness value of the retrofitted specimen was 2.34 times the value of the original one. After sevencycles of loading, specimens O_1 and O_2 practically collapsed under the axial load of the columns, due to the excessive seismic damage. Thus, the peak-to-peak stiffness values of these sub-assemblages were equal to 0.078 and 0.14 kN/mm respectively, while for the same lateral displacement of 45mm the strengthened specimens RWL_1 and RWL_2 showed 16.42 and 9.05 times higher values than columns O_1 and O_2, respectively. During the seismic tests, the stiffness values of sub-assemblages RWL_1 and RWL_2were almost similar and after eleven cycles of lateral loading (drift 6.63 percent) the columns maintained 15.7 and 20.45 percent of their initial stiffness, respectively. In the case of the control specimen, RC_2, the corresponding value was 26.49 percent. The effectiveness of the column retrofitting in improving the stiffness of the sub-assemblages was also evaluated by the values of stiffness ratios RWL_1/RC_2 and RWL_2/RC_2, which were equal to 0.613 and 0.746, respectively, at the end of the seismic tests.

In Fig. 9 the envelope curves of the original and retrofitted columns are presented. The hysteretic response of the sub-assemblages demonstrates the significant improvement in lateral strength of columns RWL_1 and RWL_2, with respect to that of the original specimens with deficient lap splices, O_1 and O_2. This is clearly illustrated in Fig. 10, where values of lateral strength of the original and retrofitted sub-assemblages are compared. In the first lower half cycle of the lateral loading the strength of column RWL_1 was 61.07 percent higher than the strength of the corresponding original specimen, O_1, while in the case of columns RWL_2 and O_2 this value was equal to 57.63 percent. For seismic actions of inter-storey drift angle, R, of 4.59 percent the strength ratio values RWL_1/O_1 and RWL_2/O_2 of the upper half cycles were equal to 17.93 and 10.46, respectively. During the incremental displacement of the lateral loading, strength values of the control specimen RC_2 were gradually increased, while a minor, mild reduction of strength was observed during the seismic tests of the retrofitted columns RWL_1 and RWL_2. Nevertheless, the strength of these columns was substantially higher than the strength of the original specimens and very close to the corresponding values of the control specimen, RC_2, until the end of eleven cycles of seismic loading. After eleven cycles of loading (drift angle R 6.63 percent), the lateral strength of structure $RWL_1$was 57.45 percent (push half-cycle) and 66 percent (pull half-cycle) the strength of the control specimen, while in the case of column RWL_2 these values were 62.8 and 88.76 percent, respectively.



(a) Original column specimen with deficient lap splices

(b) Retrofitted column via R/C jacketing and welding of the lap-spliced bars

Fig. 4 Original and retrofitted column specimens

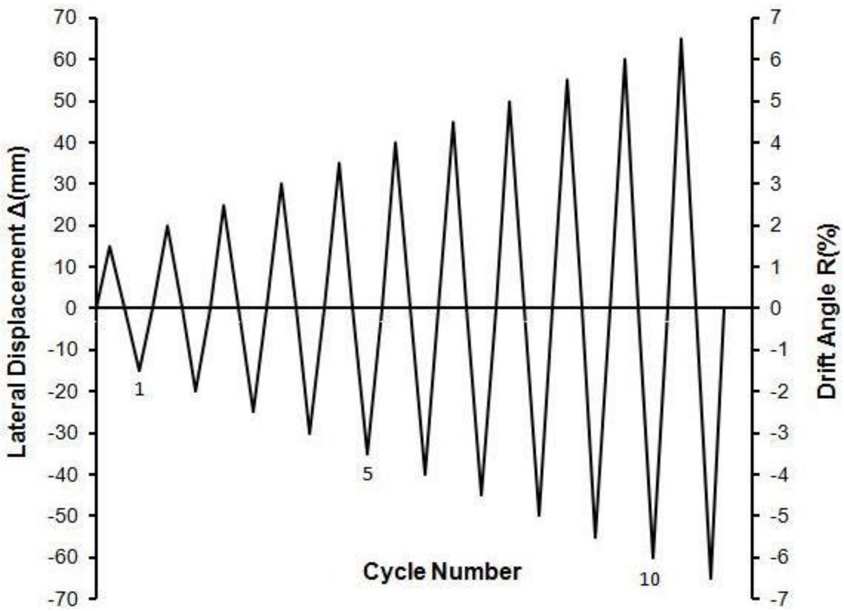
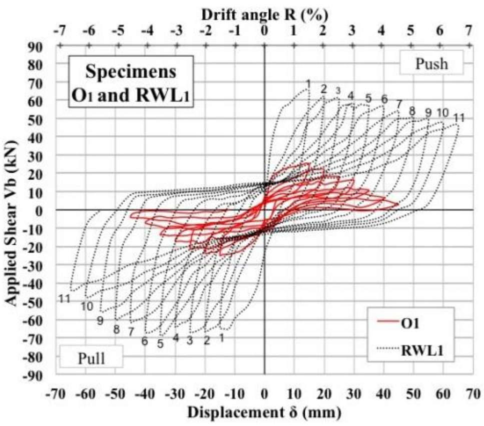


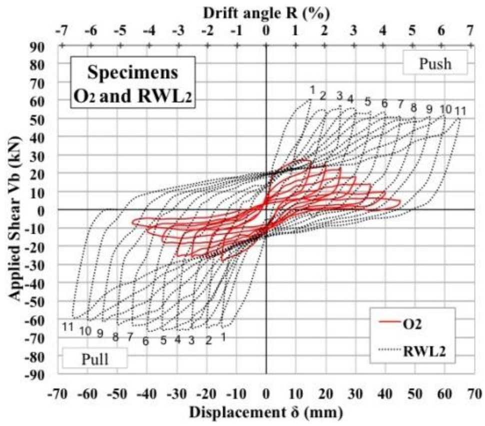
Fig. 5 Lateral displacement history

7. Failure mode of the enhanced sub-assemblages and evaluation of the strengthening process

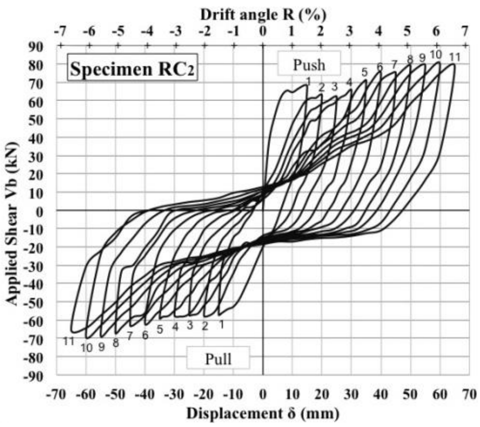
The performance of both the original and retrofitted sub-assemblages under the cyclic lateral loading is presented here and discussed in terms of load-deformation response and failure mode of each column. Figs. 6-10 clearly demonstrate the effectiveness of the retrofitting of both the columns with deficient lap splices of $20d_b$ and $24d_b$ and the specimen with continuous reinforcement in improving the overall lateral response of the strengthened sub-assemblages. The substantial improvement in the seismic behavior of columns RWL_1 and RWL_2, with respect to the lateral performance of the original specimens, O_1 and O_2, is reflected in the hysteresis loops shown in Figs. 6(a) and 6(b).



(a) Hysteretic loops of specimens O_1 and RWL_1



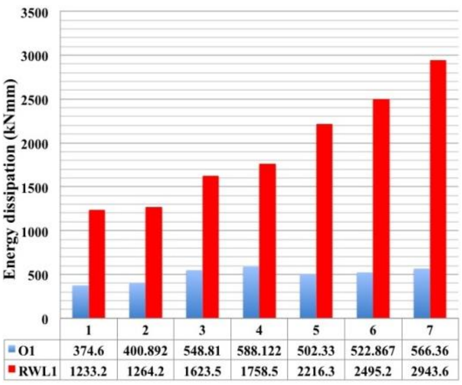
(b) Hysteretic loops of specimens O_2 and RWL_2



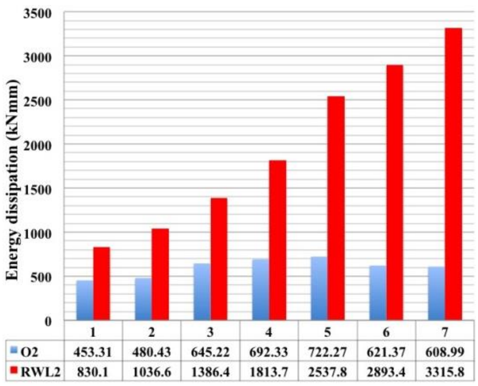
(c) Hysteretic loops of specimen RC_2

Fig. 6 Plots of applied shear-versus-displacement for the original and the retrofitted column sub-assemblages

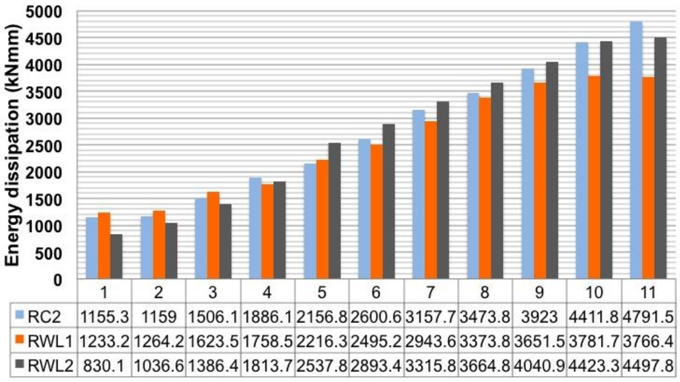
Specimens O_1 and O_2, representative of columns with substandard reinforcement details found in structures build prior to the 1960s and 1970s, performed poorly under reversed lateral deformations. Hence, the hysteresis loops of these sub-assemblages were characterized by significant deterioration of strength and peak-to-peak stiffness. Moreover, excessive slipping of the deficiently lap-spliced bars dominated the load-deflection response of the columns for seismic actions of inter-storey drift angle, R, beyond 3.57 percent. As a result, the energy dissipation capacity of the specimens was also poor. Due to the insufficient length of lap splices and low confinement, the load transfer between the column reinforcement bars was inadequate, while significant bond slip deformations were accumulated between concrete and the longitudinal bars along the splice length. As a result, the substandard columns O_1 and O_2 exhibited premature failure of the lap splices and, after seven cycles of lateral loading, practically collapsed under the loss of load-carrying capacity (see Figs. 11(d) and 11(e)). The effectiveness of the retrofitting technique applied in columns with deficient lap splices is clearly demonstrated by the spindle-shaped hysteresis loops of columns RWL_1 and RWL_2. High levels of seismic energy were dissipated in the plastic hinges of these sub-assemblages, similar to those dissipated in the case of the control specimen RC_2, with the continuous reinforcement bars in the initial column.



(a) Specimens O_1 and RWL_1



(b) Specimens O_2 and RWL_2



(c) Strengthened specimens RC_2, RWL_1 and RWL_2

Fig. 7 Energy dissipation comparison

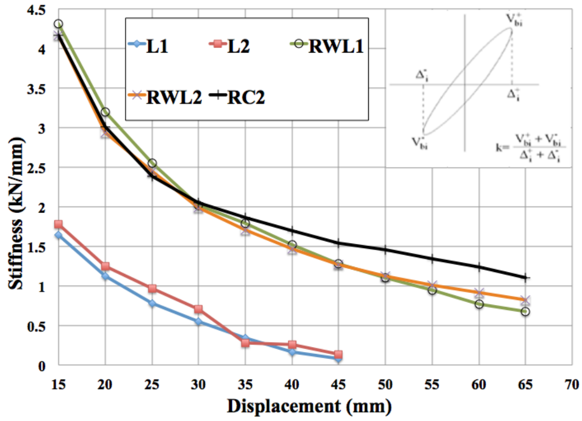


Fig. 8 Stiffness comparison

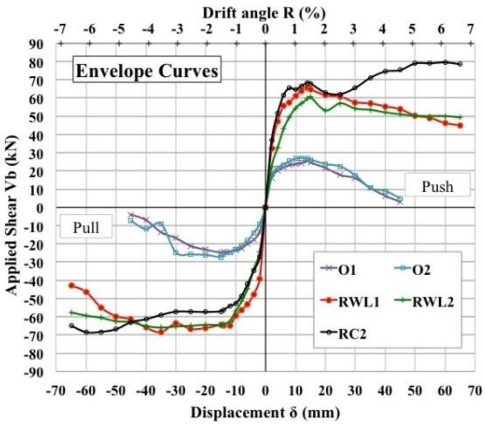
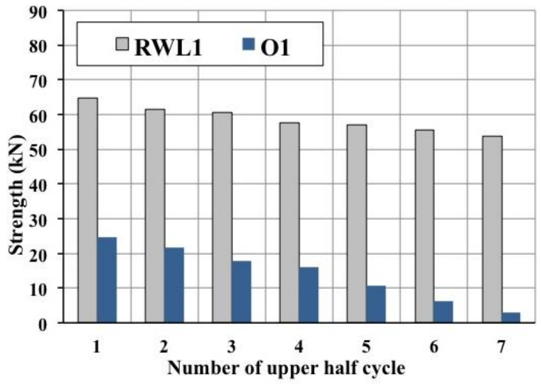
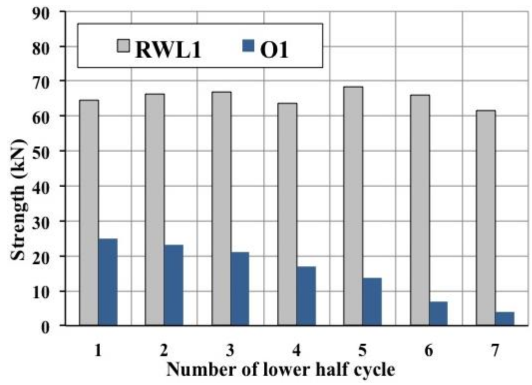


Fig. 9 Envelope curves of the original and retrofitted column sub-assemblages

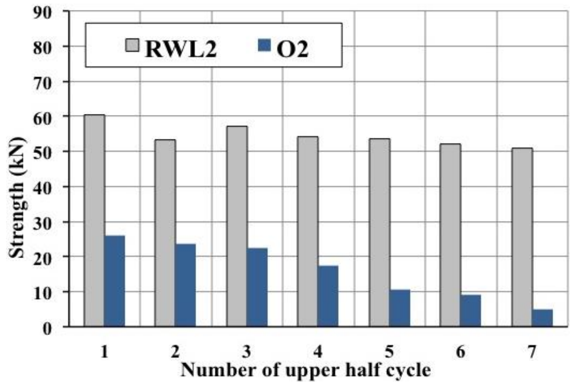
The welding of the lap-spliced bars successfully created load transfer between the spliced bars and the jacketing of specimens RWL_1 and RWL_2 provided well confinement of the column critical region, preventing bond slip deformations along the lap splices. A significant increase in strength, peak-to-peak stiffness and energy dissipation capacity of the retrofitted column specimens was observed. Hence, columns RWL_1 and RWL_2 showed a great, ductile behavior under reversed lateral displacements imposed to simulate the equivalent of strong earthquakes. Strain values of both the lap-spliced steel bars and the longitudinal reinforcement of the jackets were measured by electrical-resistance strain gauges, which were installed in the bars of each specimen. From the displacement-versus-strain diagrams (see Fig. 11) it is concluded that steel strain values of the strengthened sub-assemblages significantly exceeded the steel yielding strain in most of the cases, even for the lap-spliced bars, indicating the effectiveness of the retrofitting technique applied. Unlike the enhanced specimens, strain values of the lap-spliced bars in the original sub-assemblages O_1 and O_2 were very low, due to the excessive bond slip deformations between the concrete and the reinforcing bars along the lap splices.



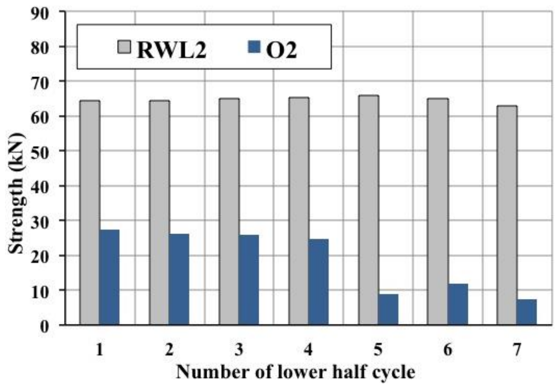
(a) Specimens O_1 and RWL_1 – Upper half cycles



(b) Specimens O_1 and RWL_1 – Lower half cycles



(c) Specimens O_2 and RWL_2 – Upper half cycles



(d) Specimens O_2 and RWL_2 – Lower half cycles

Fig. 10 Strength comparison of the original and strengthened specimens

The overall seismic behavior of the specimens, which were subjected to a large number of inelastic cycles, is also reflected in the failure mode of the columns (Fig. 12). Both the original columns O_1 and O_2 with deficient lap splice lengths equal to $20d_b$ and $24d_b$, respectively, exhibited unfavourable, brittle, premature splice failure. During the first cycle of loading hairline cracks were formed around the columns, while the first splitting cracks along the splice length started to form for drift angle, R, equal to 2.55 percent. During the incremental displacement of the lateral loading, gradual dilation of these cracks resulted in early bond failure between the concrete and the lap-spliced steel bars, loss of the cover concrete and subsequently in disintegration of the core concrete due to the decrease of the load-carrying capacity of the columns. Consequently, after the failure of the transverse reinforcement, the longitudinal bars buckled under the axial load of the specimens. Eventually, the severe accumulated damage in the critical region of the original sub-assemblages resulted in the collapsing of the columns, which were unable to withstand the vertical axial load. The enhanced sub-assemblages exhibited cracking patterns dominated by flexure. After the formation of the main flexural crack at the bottom end of the columns, a few more hairline cracks were formed along the critical height during the first three or four cycles of loading. No further damage, such as loss of the cover concrete, disintegration of the core concrete or buckling of the longitudinal reinforcement, were observed in the reinforced concrete jackets of the specimens, due to the high strength of the jackets, constructed by a premixed, non-shrink, rheoplastic, flowable, and non-segregating mortar of high strength.

Ultimately, the strengthened sub-assemblages achieved a significant improvement of their flexural strength, stiffness and energy dissipation capacity and showed great seismic performance even for large inelastic deformations. All the retrofitted columns exhibited ductile failure, with the formation of the plastic hinge in the bottom end of the columns.

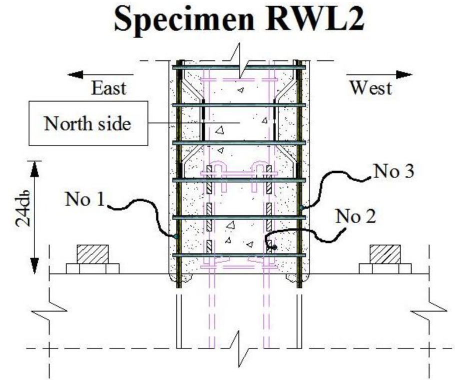
8. Monitoring of steel micro-strain

Fig. 11 illustrates the plots of displacement-versus-strain observed in each cycle of the seismic loading for the plain longitudinal reinforcement of the initial column and for the longitudinal steel bars of the jacket of sub-assemblage RWL_2 (see Figs. 11(b)-11(d)). Reinforcement details of the enhanced sub-assemblage RWL_2 and the location of each strain gage are shown in Fig. 11(a). The displacement-versus-strain diagrams for the longitudinal bars of the original columns O_1 and O_2 are also presented in Figs. 11(e) and 11(f). The location of the strain gages in the original column specimens was the same with that of strain gage No2 in column RWL_2.The premature lap splice failure in the original columns O_1 and O_2 resulted in very low strain of the longitudinal steel reinforcement bars. The plots shown in Figs. 11(e) and 11(f) indicate that both the original columns with lap splices performed poorly under the imposed seismic lateral loading.

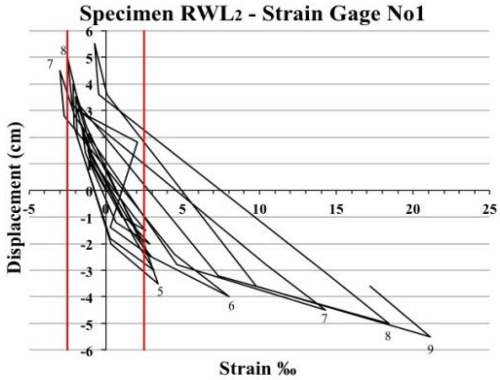
Strain values of the longitudinal bars of column sub-assemblage RWL_1 were very close to the steel yield strain and continued to increase during the incremental displacement of the lateral loading. This increase in strain values indicates that the continuity of the lap-spliced reinforcement bars was successfully created and bond slip deformations between concrete and the bars were prevented (Ehsani and Wight 1985). Strain values higher than yield strain were also observed in the case of specimen RWL_2 for both the reinforcement bars of the jacket (strain gages No1 and No3) and for the plain lap-spliced bars of the initial column (strain gage No2).The continuous increase in steel strain, observed in specimens RWL_1 and RWL_2, for both the lap-spliced bars of the existing columns and the longitudinal reinforcement of the jackets, reflects the effectiveness of the retrofitting technique in improving the load transfer between the deficiently lap-spliced bars.

9. Conclusions

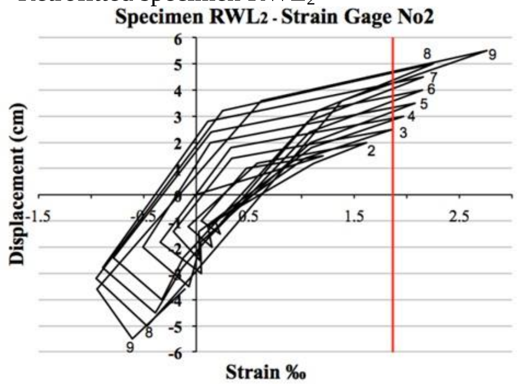
In this study, an experimental program was conducted for eight 1:1.5 scale reinforced concrete rectangular column specimens to identify the feasibility and performance of a pre-earthquake retrofitting technique applied in substandard columns with deficient lap splices. Five original specimens representative of those found in pre-1970 structures, one with continuous longitudinal reinforcement, two with lap splices of $20d_b$ length and two more with lap splices of $24d_b$ length, were constructed. One of the column sub-assemblages with lap splices of $20d_b$, one with lap splices of $24d_b$ and the column with the continuous longitudinal bars were strengthened by reinforced concrete jacketing, while the lap-spliced bars of the old columns were welded.



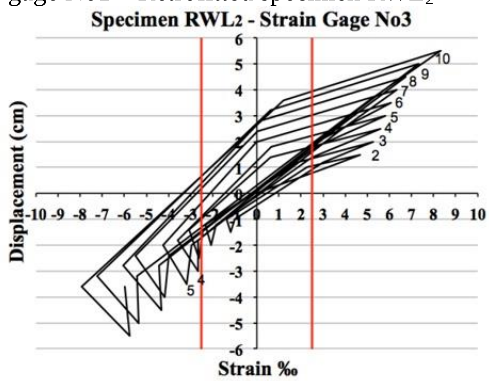
(a) Position of the strain gages (North column side) Retrofitted specimen RWL_2



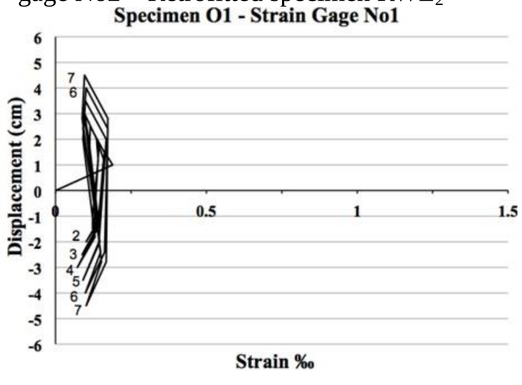
(b) Displacement-versus-strain measured by strain gage No1 – Retrofitted specimen RWL_2



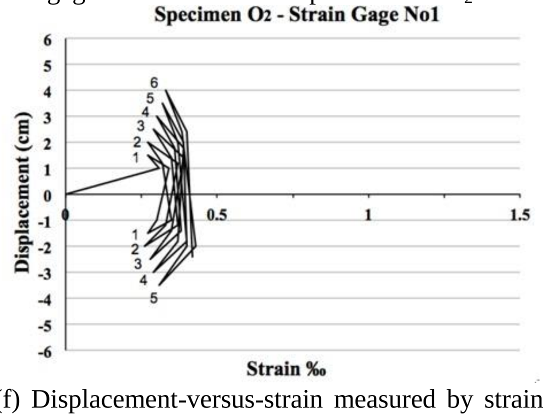
(c) Displacement-versus-strain measured by strain gage No2 – Retrofitted specimen RWL_2



(d) Displacement-versus-strain measured by strain gage No3 – Retrofitted specimen RWL_2

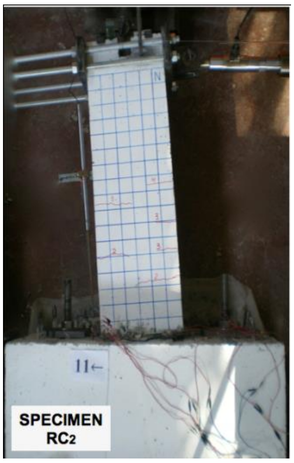


(e) Displacement-versus-strain measured by strain gage No1 – Original specimen O_1

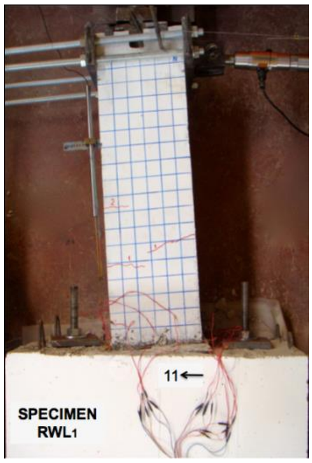


(f) Displacement-versus-strain measured by strain gage No1 – Original specimen O_2

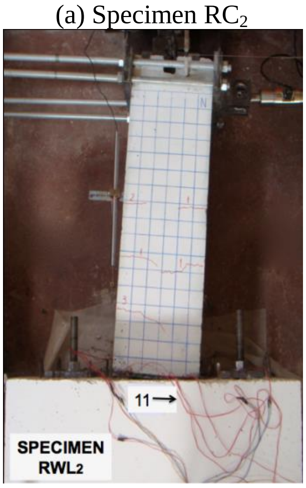
Fig. 11 Plots of displacement-versus-strain of the longitudinal reinforcement for the retrofitted specimen RWL_2 and the original columns O_1 and O_2



(a) Specimen RC_2

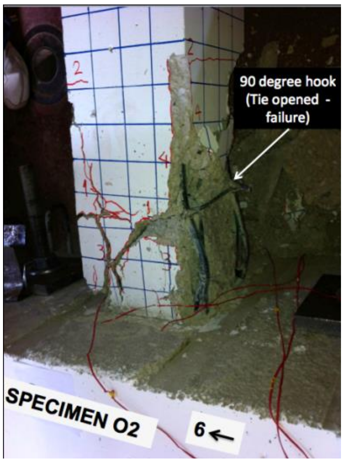


(b) Specimen RWL_1



(c) Specimen RWL_2

(d) Specimen O_1



(e) Specimen O_2

Fig. 12 Post-damage views of the collapsed subassemblages (a) RC_2, (b) RWL_1, (c) RWL_2, (d) O_1 and (e) O_2

The two original specimens, with lap splices of $20d_b$ and $24d_b$ and the three retrofitted column sub-assemblages were subsequently subjected to a large number of reversed lateral displacements to simulate the equivalent of strong earthquakes and the seismic performance of the structures was compared.

• The substandard original columns O_1 and O_2 performed poorly under the cyclic lateral loading and exhibited brittle premature failure of the lap splices. The load-deflection response of these columns was dominated by bond slip and the specimens finally collapsed under the axial load, due to the loss of load-carrying capacity.

• A significant improvement in the flexural strength, peak-to-peak stiffness and energy dissipation capacity of the retrofitted specimens with welded lap splices, RWL_1 and RWL_2, with respect to the lateral response of the original columns, O_1 and O_2, was observed during the seismic tests.

• Specimens RWL_1 and RWL_2 performed very satisfactorily during the cyclic loading sequence to failure, showing a seismic behavior similar to that of the control specimen RC_2 with continuous longitudinal reinforcement in the old column.

• The welding of the lap splices effectively created the continuity of the longitudinal spliced reinforcement, while the confinement provided by R/C jacketing further improved the load transfer between the spliced bars and prevented the significant bond slip deformations between concrete and the steel bars along the splice length. Thus, the strengthened specimens exhibit a desirable ductile failure in pure flexure, with the formation of the plastic hinges in the bottom end of the columns.

• The lap splice lengths of $20d_b$ and $24d_b$were both deficient for transferring the significant tensile forces between the bars. This is reflected in the failure mode of the original sub-assemblages. Nevertheless, the retrofitted structures with lap splices of both $20d_b$ and $24d_b$ length in the old column showed great performance under large inelastic cyclic deformations.

• Ultimately, the retrofitting technique applied, proved to be an effective method for the pre-earthquake strengthening of columns with deficient lap splices.

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