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INNOVATIVE PROCEDURES TO REHABILITATE SEVERELY DAMAGED CONCRETE COLUMNS AND COMPARISON WITH ALTERNATIVE METHODS

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Abstract

Seismic retrofit techniques for strengthening of damaged concrete members offer interesting approaches when innovative materials or devices are used. Laboratory program was conducted on tests o full scale reinforced concrete columns retrofitted with new techniques after severe damage suffered. CFRP sheets and anchors were used for strengthening or for creating ductile elements or adding new shear reinforcement as a jacket for columns. Mechanical splices were used to provide continuity to the reinforcement, and to replace buckled bars in location where concrete has crushed and the bars buckled. It was found that performance of the retrofitted column hinges was comparable to those using conventional techniques and the feasibility depends on the degree of damage, the cost of replacement, and performance required. The mechanical splices developed a well behavior under the axial load and moments effects by lateral cycle loadings applied to the columns. The CFRP materials confining the hinge region of the column provided large ductility to this member. This CFRP sheets and anchors allowed carrying out higher shear loads than the original column and wall tested. The performance in shear capacity and ductility of the rehabilitated column hinges using the innovative procedure were compared with other alternative techniques of retrofit, remarking the effectiveness of the innovative rehabilitation approaches.

Keywords: Column Hinges, Mechanical Splices, Carbon Fiber Reinforced Polymer CFRP, CFRP jacketing, CFRP anchors.

1. Introduction

Older structures and poorly constructed or badly designed structures are likely to be damaged in an earthquake. Using innovative materials or devices for strengthening of reinforced concrete members offers interesting approaches. The cost-benefit of the retrofit procedures can be convenient for the stakeholders of the infrastructure to be rehabilitated. To repair may be less expensive than replacing if it is compared the cost to demolish and rebuild a new column with repairing the damaged area of the structural member. This paper is aimed to show the seismic rehabilitation of severely damaged columns with nonconventional procedures and their behaviors, and compare these responses with repaired damaged columns which were rehabilitated using alternatives methods. Laboratory program was developed to test and repair concrete column hinges using Carbon Fiber Reinforcement Polymer CFRP and mechanical splices. Then backbone curves obtained from the cycle behavior of the tested columns were compared with other types of retrofit methods listed at the background. The intent is to provide data that designers can use to make decisions for rehabilitation of structures to meet performance based seismic design requirements.

2. Background

2.1 Study cases of retrofit using jacketing

Aboutaha[1] studied the improving of short length of lap splices length in concrete columns using steel jackets. One of these columns (FC-17) had square cross section of 18"x18, 8 #8 longitudinal bars, lap splice length was 24in. and #3"@16" stirrups. It was used ¼" thick steel, steel angles at the corners attached to the surface by two epoxy-grouted steel bolt anchors were added on one face only as Fig.1 shows. Aboutaha found that the steel jacket resulted in large improvements in the deformation capacity, more than 5% drift ratio, with little reduction of strength and stiffness. Flexural crack and diagonal shear cracks on the column appeared above the steel jacket. The steel jacket was removed to inspect the region of the splice area and no major damage was found.

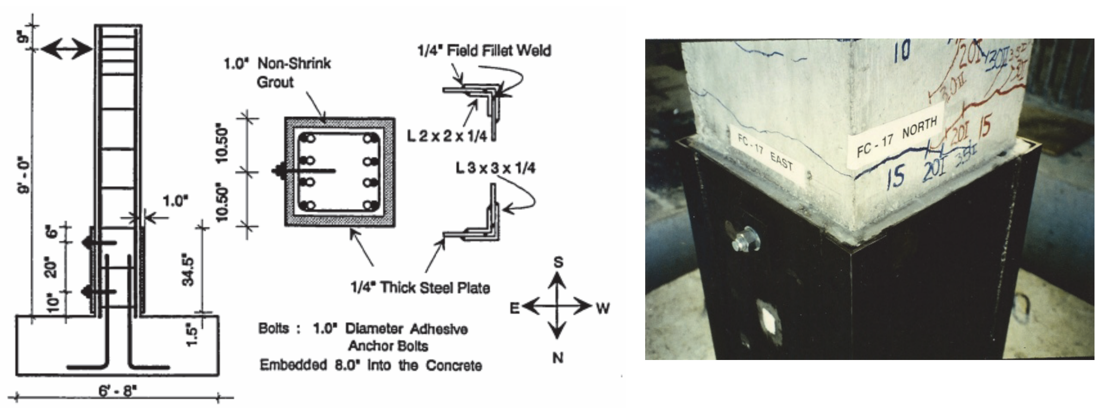

Fig. 1 –Steel jacket for Column FC-17 (left), and view of the retrofitted column after the test (right).[1]

Kim[2] researched poorly detailed lap splices and insufficient confinement in concrete columns, and developed further rehabilitation procedure using Carbon Fiber Reinforced Polymer (CFRP). Fig.2. shows column 2-A-S8-M which had an 18in x 18in cross section and 8 longitudinal #8 bars with #3 ties @ 16in. For the as-built test, the lap splice failure of the side in tension resulted in sudden drop of the lateral load. Horizontal cracks appeared on the damage side and vertical cracks along the lap splices appeared at the failure of the splice. The test results for the retrofitted column indicated increase strength of the column, and also ductility. It was also observed that capacity of column before and after retrofit reached the same lateral deformation in both directions was nearly the same, indicating the effectiveness of the CFRP jacket.

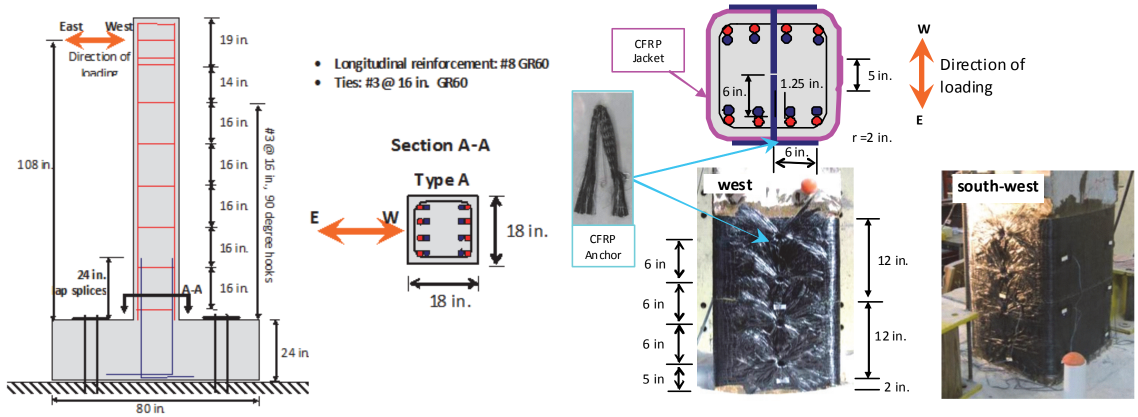

Fig. 2 – Elevation and plan view of the column previous to the retrofit including Lap Splices (left), and Cross Section of the 2-A-S8-M with details of the CFRP material applied (right).[2]

2.2 Study case of retrofit using CFRP for longitudinal reinforcement and jacketing

He[3] studied the behavior of damaged columns also. The column 2-R was repaired using CFRP sheets vertically placed to act as longitudinal reinforcement and CFRP jacket were added for confinement. For anchorage was used steel bolts to anchor the longitudinal CFRP sheets to the base. The square section was 22in x 22in, with 4#9 and 8#8bars. Ties were with #3 square and with octahedral shapes spaced @ 3.25in. Details of the anchorage used for the longitudinal CFRP sheets are shown in Fig.3. The bolted plates were used on the north and south faces of the column and U-anchors at the end of the strips were embedded in grooves at the bottom of the column. This column was tested under a constant axial load (150kips) and cyclic lateral load and repaired afterwards. Compressive strength of the concrete was 5.8ksi. During the test, the U-anchors on the east and west sides of the column pullet out. The damaged hinge zone of the retrofitted column was located 20in above of the base. Rupture of the CFRP was also observed during test near the rounded edge for the anchor plate on the north and south faces.

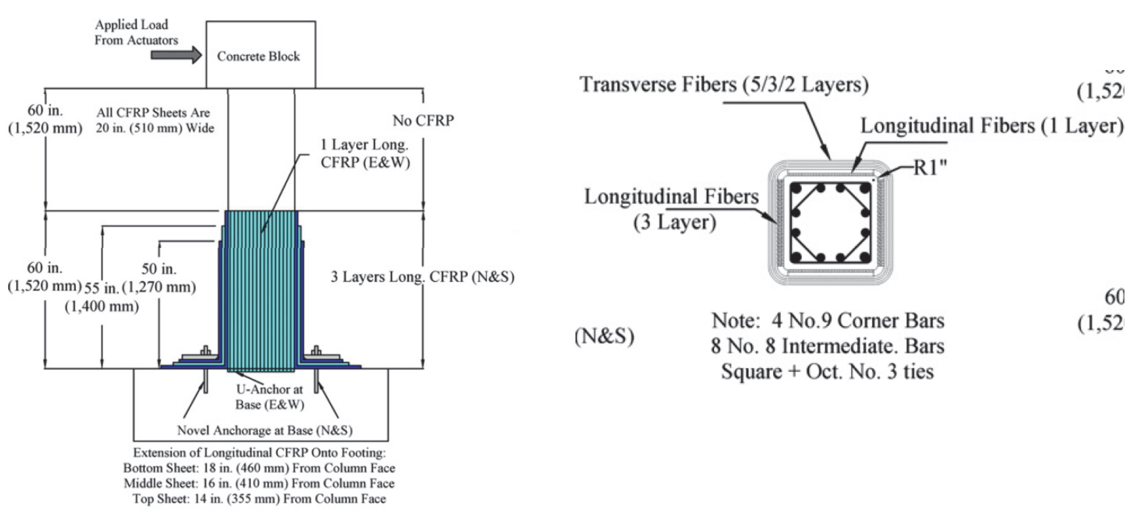

Fig. 3 – Elevation of the column retrofitted with longitudinal CFRP and CFRP jacket (left) and cross section of the Column 2-R with details of the CFRP material applied (right).[3]

3. Laboratory Program

Two reinforced concrete columns (RC-1 and RC-2) were tested under lateral cyclic loads and constant axial loads until the failure. The columns had 16in x 16in cross section, 116in tall, and 8-#8 longitudinal bars with three bars in each face, column ties were #3 @ 6" with 90 degrees hooks. The nominal concrete strength was f'c = 3ksi [4]. The test setup was made to produce double curvature deformation, the bottom support of the specimens was fixed, and at the top the rotation was restrained and free for lateral displacement as is shown Fig.4 [5]. It was assessed the condition of each of the two formed hinges of the 2 columns having a total 4 different hinges to rehabilitate.

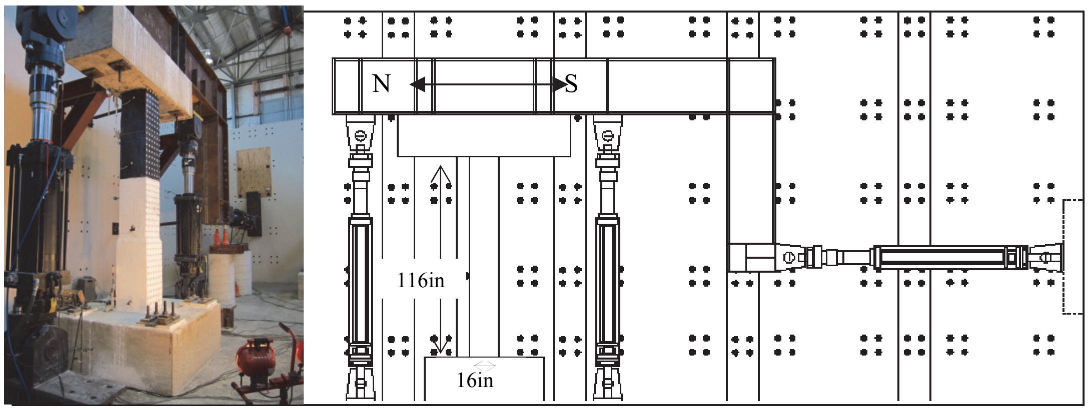

Fig. 4 – Test setup for the fix bottom – top rotation restrained specimen

3.1 Retrofit of first group of hinges from Concrete column RC-1

For the first column severe damage occurred at both ends with buckling of longitudinal bars and crushing of the concrete at the bottom and spalling of the cover at the top. The constant axial load applied was 150kips. The top end of the column where spalling of the cover but no buckling occurred, loose concrete was removed and mortar was used to replace the broken concrete (Huaco[7].) Fig.5 shows the damaged top and bottom hinges and the retrofit process. The damaged regions were wrapped with 2 layers of CFRP sheet (0.04in thickness each layer) at the first 24in and some intermediate CFRP anchors were installed midway between the corners. For a second 24in length was used one jacket only. At the bottom of the column, short mechanical splices were use to replace the buckled bars and to provide continuity to the longitudinal bars and cold joint was used at inferior and superior set of splices, details of the retrofit procedure is at Huaco and Jirsa[5]. This retrofitted column was tested using the same deformation pattern and constant axial load than the original.

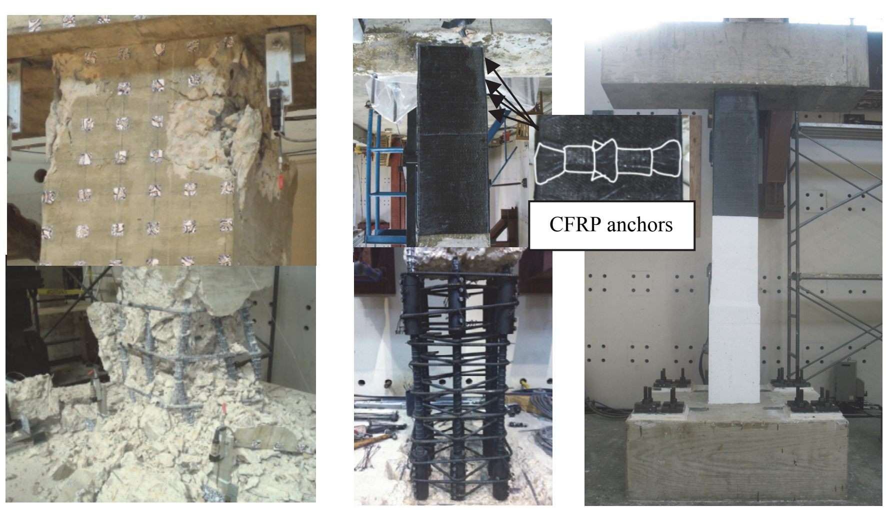

Fig.5. First column, damaged condition and retrofit

3.1.1 Global behavior of the retrofitted column RC-1R

The behavior of the member finished was nearly linear up to lateral displacement of 1.66in or 1.43% drift ratio. The shear force to the North was 46.85kips, and to the South was -38.55kips.The strain of the new bars on the bottom of the column reached yield value for Grade 60 steel. The maximum shear capacity was 60.9kips North and 56kips South. The lateral deformation was 2.80% drift ratio in both directions. Degradation of stiffness and strength was seen at 5.5% drift ratio both directions. Pinching effect seemed to be appeared for pushing loading when the lateral load was dropped as can be seen in Fig.6. The cracks appeared near the bottom became wider, while on top the CFRP started to debond. The specimen was not able to sustain the 150kips axial load in the 9th cycle and the test was stopped when the lateral load dropped by 30kips. Shear failure developed in the existing concrete above the mechanical splices zone as can be seen in Fig.7. The axial load was reduced in order to increase the lateral deformation noticing an extra capacity of the bottom half of the column in shear and displacement. Fig.6 shows the behavior of the as-built column RC-1 and after retrofitting RC-1R. Values of displacement are normalized by drift ratios. It was observed that RC-1R had less stiffness than the as-built column, even though the retrofitted column had a larger cross sectional area at the bottom. This was more evident when the column was loaded in the south direction. Since large cracks opened at top and bottom of the column, it can be assumed that the existing bars were deformed more than in the original test. RC-1R had slighter higher lateral capacity than RC-1 and values of ultimate drift ratio are comparable.

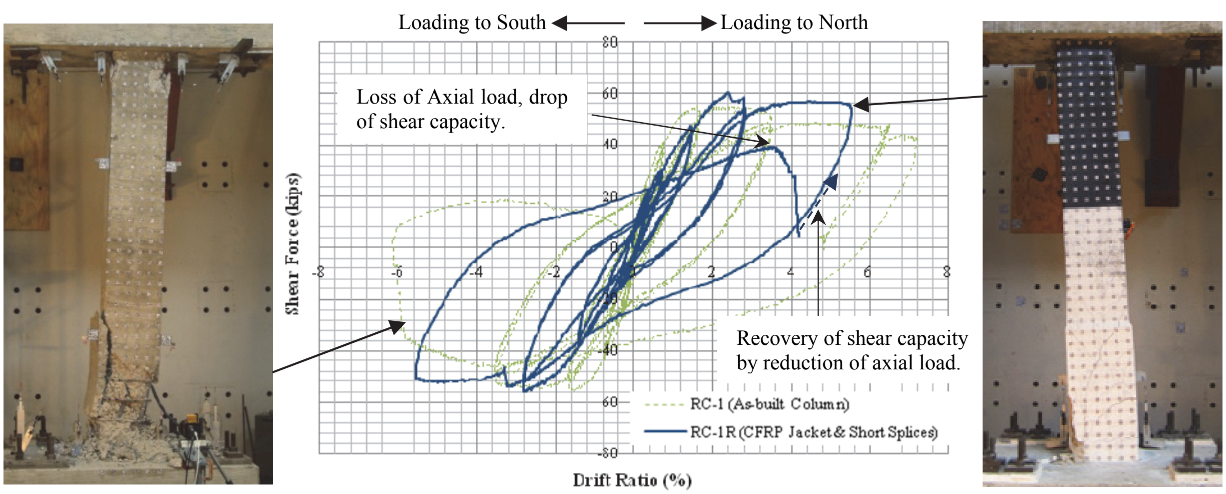

Fig.6. Shear force vs. drift ratio of RC-1 and RC-1R

3.1.2 Behavior of the bottom hinge region with short mechanical splices: RC-1R Bottom Half

For tension stress the splices behaved linearly however for compression the splice behaved nonlinearly. Both the superior and inferior set of splices shown similar responses under the axial load induced through the flexion by the lateral load applied to the specimen. It can be also appreciated that the strain for the inferior set next to the column base is larger than the superior set next to the as-built concrete section. Since the flexural moment is biggest at the base of the column, it was expected to have larger strain for the splice located in the inferior level. Those longitudinal bars worked under tension stress since the neutral axis at that level of the column (7in above the base) moved to the south, making all the bars in the section work for tension stress and having the 2.1in concrete cover working under compression stress. The transverse reinforcement around the mechanical splices and new longitudinal bars confined the core and allowed the column to reach large ductility. Those bars were in tension through the loading history with strains increasing as the lateral deflection increased. The largest strain measured was 0.0011. The hinge zone appeared where the failure of the as-built column occurred. Fig.7 shows the comparisons of the response of the as-built column RC-1 and the bottom half of the retrofitted column RC-1R bottom half (short mechanical RC-1R Bottom Half splices). The stiffness of RC-1R bottom half under loading on both directions was slightly less than RC-1 even though the clear cover was increased but the ultimate drift ratios were also comparable.

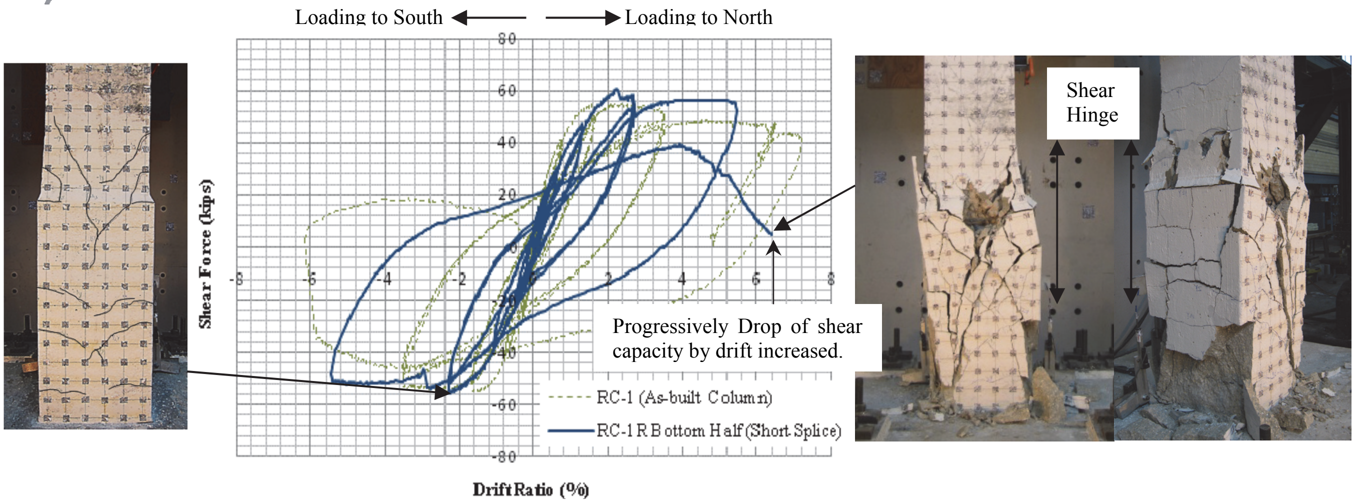

Fig.7. Shear force vs. drift ratio of RC-1 and RC-1R Bottom Half (center), diagonal cracks forming the shear hinge (left), and condition of specimen at failure: cracks pattern producing the spalling of the 2in cover (right)

3.1.3 Behavior of the top hinge region with CFRP jacketing: RC-1R Top Half

Slighter higher shear forces were observed in RC-1R top half but the stiffness at low drifts was greater for the retrofitted column, especially the specimen was loaded to the south. Loading-to-North curve is stiffer than Loading-to-South because the presence of the CFRP anchors in the south face of the specimen, see arrows above and below curves at Fig.8. When the specimen is loaded to north, the CFRP anchors is in the compression region of stress, the anchors provided compression strength capacity. However, the specimen is loaded to the opposite side, the CFRP anchor is in the tension stress region. The anchors do not provide important tension capacity to the section. Fig.8 shows the response of RC-1 and RC-1R top half (CFRP jacket). Lateral displacements are normalized by drift ratios. CFRP jacket was removed after the test and it was noticed that CFRP interior part of anchor were not pulled out and the condition of concrete core had no severe damage (Fig.8). Since the failure of RC-1R occurred on the bottom half, and the top half was in good condition, it was concluded that this top hinge had the capability to perform larger lateral displacement therefore dissipate more energy. The maximum strain reached on the 2 layers CFRP jacket was 0.0038; and 0.0014 on the 1 layer CFRP jacket. Strains in most of the longitudinal bars in the top of the column yielded reaching max strain of 0.0042.

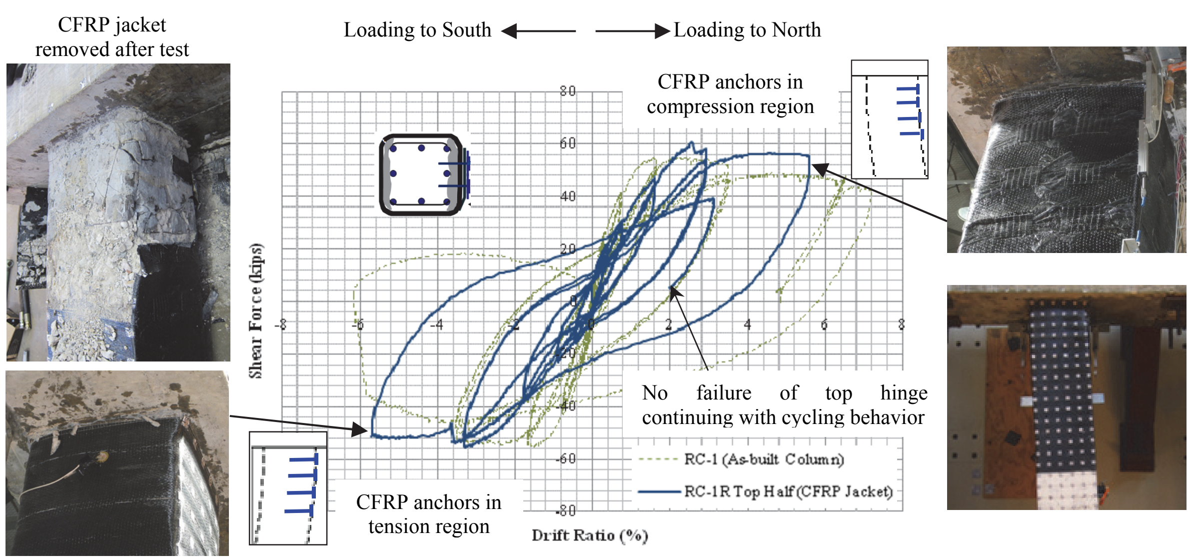

Fig.8. Shear force vs. drift ratio of RC-1 and RC-1R Top Half

3.2 Retrofit of second concrete column RC-2

The constant axial load applied was 350kips (Huaco[7]). Axial failure was reached with the column exhibiting severe deterioration at both ends and buckling of longitudinal bars coupled with crushing of the concrete as it seen in top corners of Fig.9. The repair procedure consisted of replacing the damaged portion of the column with new bars and higher strength concrete. Mechanical splices were used to join the old with the new bars. The column was divided into two parts to be tested as cantilever columns by applying the lateral load at the middle height of the original one, having single curvature deformation which is the approximate the condition of one of the half parts of as-built double curvature deformation of RC-2. The height of each cantilever was 58in. The two hinges were tested separately as cantilever columns with no axial load applied, considering the worse tension condition for the splice to be tested inside the hinges. One half of RC-2 was repaired using short mechanical splices (6.8in length) RC-2R-SMS, and the second half was used long mechanical splices (10in length) RC-2R-LMS. Details of the rehabilitation procedure are in Huaco and Jirsa[6]. Fig 9 shows the damaged hinges, cutting the buckled bars leaving a protrusion for the application of the mechanical splices and the old and new bars gathered with the mechanical splices are shown. Cold joint was used at base. Because larger radio of splice, the concrete cover increased to 2.1in from edge of longitudinal bar, increasing the cross section to 18in x 18in. Confinement region area at splice height increased but not at bars height. The protocol of load for the test of both columns was following the procedures of FEMA 461 until 2.10% drift ratio which is above the maximum allowed drift recommended in the ASCE07-10 seismic design provisions.

Fig.9. Two new cantilever specimens from RC-2. Application of short mechanical splice (left) and long mechanical splice (right).

3.2.1 Behavior of the hinge region RC-2R-SMS

Flexural cracks developed at 0.55% drift or 0.30in (lateral displacement) in both faces of the column at the base. The lateral load was 32kips. One crack with of 0.027in was measured at the cold joint. Yielding occurred at 1.1% drift ratio with a lateral load of 50kips in the north direction Flexural cracks propagated through the column. The crack at the base opened to 0.05 in. The strain measured in the mechanical splices was 0.0009 in tension and 0.00015 in compression. The longitudinal bars strain was between 0.0025 in tension and 0.0001 in compression. No concrete spalling was observed at this point and none of the ties reached yield, pinching and larger energy dissipation was observed also, and there was a slight degradation of stiffness and strength too. When lateral deformation reached 1.40in, the shear capacity dropped 60kips to 47kips after a loud noise was heard because one of existing longitudinal bars fractured at the south-east corner of the specimen. Further increases in deformation resulted in failure of a second bar produced at 53kips and 2.0in lateral displacement. The fractured existing bar was located at the center of the three bars at tension region of the column. The response was comparable for both Loading-to-South and Loading-to-North. The test continued until the rupture of 3 existing longitudinal (Fig.10). The maximum deformation reached was 3.62 in obtaining an ultimate load of 9.5kips before last bar fractured. It was present lifting as result of the rupture of bars.

Fig.10. RC-2R-SMS at finishing of the test (left), new longitudinal bar broken and lifting at base (right)

3.2.1 Behavior of the hinge region RC-2R-LMS

No crack appeared at 38kips loads, the specimen behaved also in linear range at 0.60in lateral displacement (1.05% drift ratio). The lateral load to south direction measured was 46.17 kips and -46.87 kips loading to north. Degradation of shear capacity was also noticed, measuring drop of 3kips. Pinching effect was observed too. Lifting appeared lightly at the cold joint lifting 0.066in. and no spalling was presented despite that the cover was taking all the compression internal force by the flexural load. The splices were in tension deformation indicating that the neutral axis was located in the concrete cover zone. The ties neither the long mechanical splices did not reach the yield, however the new longitudinal bars reached the yield having 0.0030 strain in tension and 0.0007 in compression. Further loading, pinching and degradation of shear force were noticed at horizontal force of 60kips and 1.20in lateral displacement was 60kips for both direction of loading. The maximum shear capacity was 71.65 kips and the lateral deformation was 4.72in. The lifting at north measured was 0.64in. The cracks propagated from face to opposite face of the column and spalling of the concrete cover happened. The test continued loading to north and it was hear a loud noise when the lateral displacement of column 2.96in the load dropped from 61.40kips to 47.40kips, it was noticed that one existing longitudinal corner bars fractured. The internal area between the curves of the hysteretic loops became larger indicating mayor energy released at comparison than previous loops correspondent for the linear behavior of the specimen. Loading to south it was noticed the rupture of one existing longitudinal bar and the test stopped. This failure was noticed by the loud noise as well. The cover of the south face until 15in above the base spalled. The lifting increased considerately (Fig.11). The lateral shear force prior the bar's fracture was 68.05kips dropping to 51.99 kips. The lateral displacement prior the rupture was 7.62in jumping to 7.68in. Energy was released in mayor amount at comparison than other hysteretic loops.

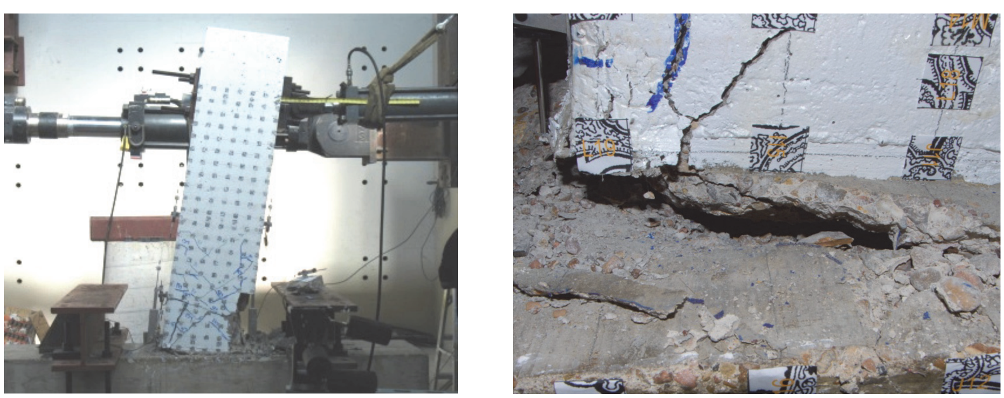

Fig.11. RC-2R-LMS at finishing of the test (left), new longitudinal bar broken and lifting at base (right)

3.2.3 Cycle behavior of RC-2R-SMS and RC-2R-LMS

Fig.12 shows a comparison of behavior for both top and bottom retrofitted hinges of RC-2. Values of lateral displacement were normalized with drift ratios. It can be seen that both halves had the same behavior in the linear range. However, the half retrofitted with long mechanical splices reached higher lateral capacity and developed larger lateral displacement. The end point bolt of the short mechanical splice in RC-2R-SMS produced the rupture of the existing bar at lower drift ratios than in RC-2R-LMS, which had long mechanical splices with rounded bolt and fracture outside of the mechanical splice. Fig. 12 shows too the type of rupture of each type of mechanical splice [6]. and the condition of the intact core for RC-2R-LMS after removing of the cover, and the spalling of cover and the final condition of RC-2R-SMS after the rupture of the three existing longitudinal bars.

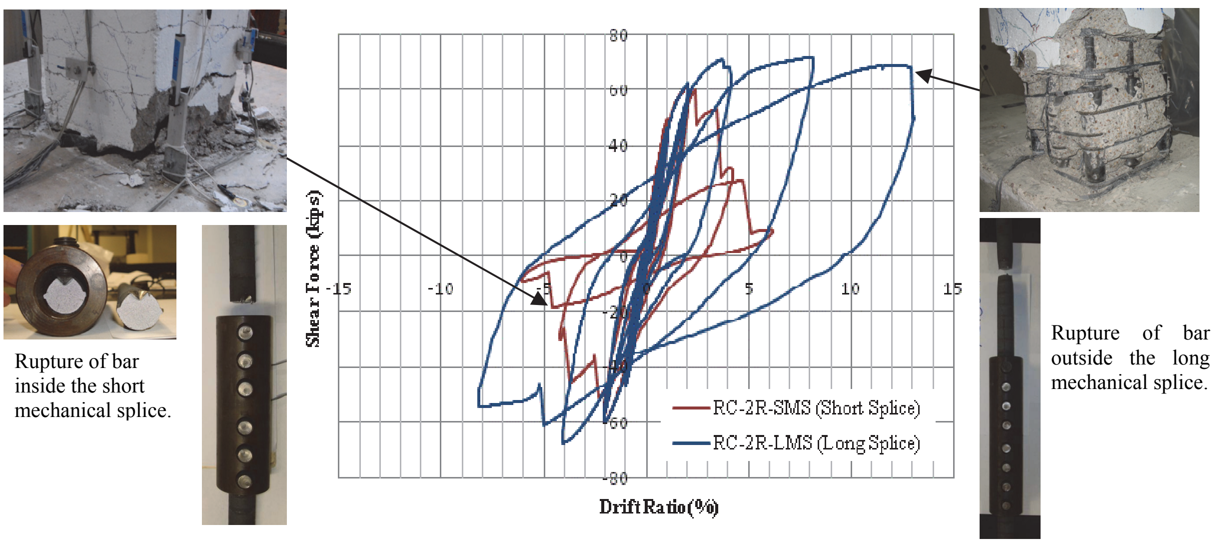

Fig.12. Shear force vs. lateral displacement of RC-2R-SMS and RC-2R-LMS

3. Comparison with other rehabilitation techniques -

It is compared the effectiveness of the retrofitted techniques explained at the background with the innovative techniques using CFRP materials and mechanical splices explained at this current paper. It is used the cycle behavior of the hinges from the specimens FC-17 [1], 2-A-S8-M [2] and Column 2-R [3]' and RC-1R Bottom Half, RC-1R Top Half, RC-2R-SMS and RC-2R-LMS. The shear capacity vs. lateral displacement responses were used to construct backbone curves following the procedures of ASCE41-13 [5,8]. The value of the measured experimental lateral capacity is normalized using the nominal lateral capacity of the as-built confined section of each retrofitted or existing column. To calculate the nominal capacity, the measured dimensions and material properties were used. Details of the calculations can be found at Huaco[7]. The lateral displacement was normalized calculating the drift ratio for each column case.

Fig.13 shows the comparisons among the retrofitted column cases with the same axial load applied P=150kips. It is appreciated that the retrofitted column with the short mechanical splices and also the retrofitted column with the CFRP jacketing were considerable more efficient than Column 2-R with higher normalized lateral capacity. It is also shown in Fig.13 than the drift ratio of Column 2-R is shorter than retrofitted column RC-1R Top Half, RC-1R Bottom Half. The hinge from RC-1R Top Half and RC-1R Bottom Half had better efficient compared with the proposed with Column 2-R. It can be explained since Column 2-R replaced important part of its core with repair mortar instead of new concrete. This mortar made Column 2-R to have a weak core section despite the two direction CFRP jacket around the hinge zone of this column. Besides, it is expected to have larger ductility with RC-1R-Top Half since the collapse of RC-1R was presented at the bottom half finishing the test.

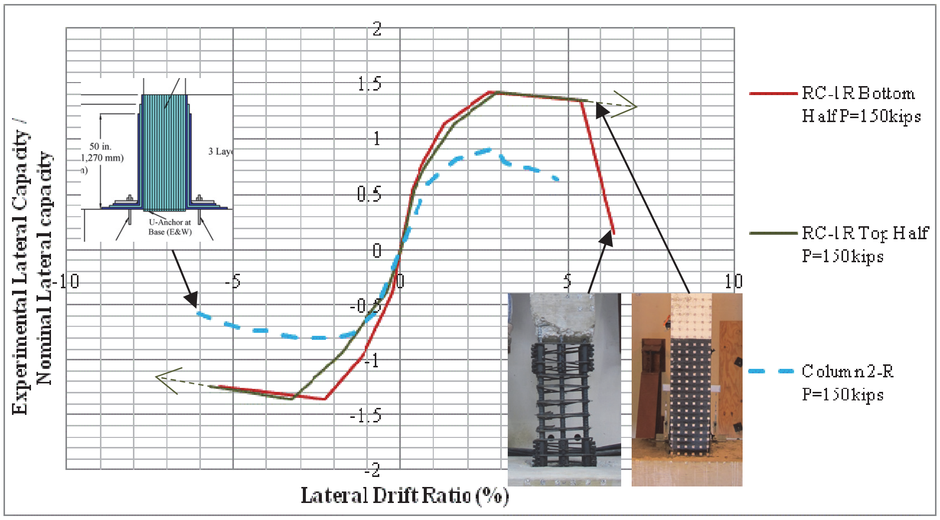

Fig.13 Comparison of the normalized ASCE41-13 backbone curves among retrofitted columns with same axial load applied P=150kips

Fig.14 shows the retrofitted column cases with no axial load applied. Normalized backbone curves of RC-2R-SMS and RC-2R-LMS have same stiffness at comparison with the retrofitted with the CFRP jacket on 2-A-S8-M or steel jacket on FC-17. However, these retrofitted columns using the mechanical splices reached larger value of drift ratio and higher normalized lateral load capacity.

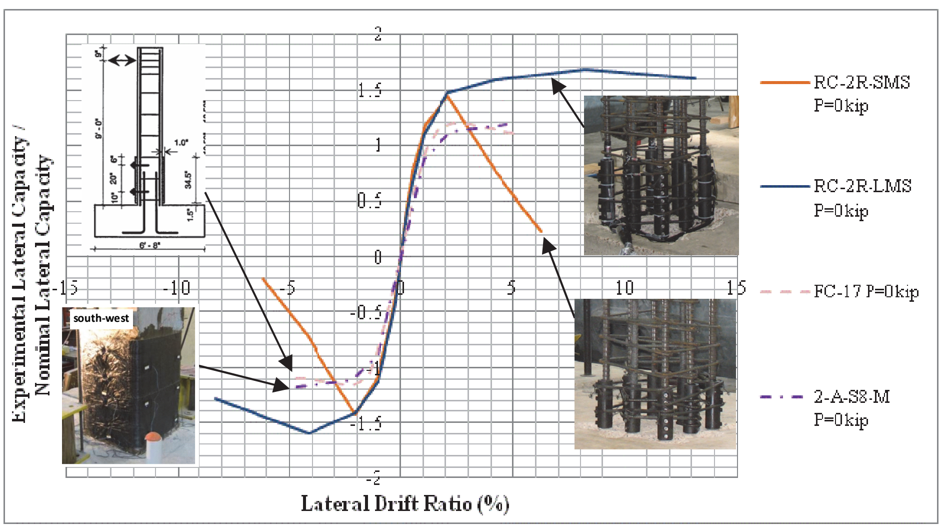

Fig.14 Comparison of the normalized ASCE41-13 backbone curves among retrofitted columns with no axial load applied P=0kip

The Table 1 shows the values of maximum shear force and the lateral displacement for case base for the normalization of shear force and calculation of drifts. Table 2 shows peak values of normalized lateral load capacity and percentage of drift ratios. It is noticed the effectiveness of the procedures proposed in this research using the innovative materials and devices as the CFRP and the mechanical splices. These material and devices were applied in appropriated location on the hinge zones of the column. This procedure made the retrofitted columns reach high performance in shear force and lateral deformation under constant axial load and cycle lateral loads.

Table 1 – Summary of maximum values of shear forces and lateral displacement by ASCE41-13

Column Hinge	**ASCE41-13**	
	Vmax (kips)	**Max Disp. (in)**
FC-17	33.00	5.45
2-A-S8-M	33.88	5.11
Column 2-R	55.00	8.00
RC-1R Bottom Half	58.45	3.15
RC-1R Top Half	58.45	3.30
RC-2R-SMS	61.28	3.62
RC-2R-LMS	71.08	7.62

Table 2 – Summary of peak values of normalized shear capacity and drift ratios for each backbone curve ASCE41-13

Related with as-built nominal capacity of specimen.

Related with height of specimens.

Column Hinge	**Calculated Nom.Lat.Cap.**	**Exp.Lat.Load / Nom.Lat.Cap.**	**Drift.Ratio %**
FC-17 P=0kip	27.50	1.19	5.05
2-A-S8-M P=0kip	28.70	1.18	4.73
Column 2-R P=150kips	60.30	0.91	4.70
RC-1R Bottom Half P=150kips	41.20	1.42	6.38
RC-1R Top Half P=150kips	41.20	1.42	5.60
RC-2R-SMS P=0kip	42.20	1.43	6.25
RC-2R-LMS P=0kip	42.20	1.68	13.14

5. Conclusions

The research represents a unique project for efficient rehabilitation of severely damage concrete members with attractive cost-time benefits. The rehabilitation was based on the use of innovative materials and devices.

The innovative rehabilitation methods resulted in members with equal or better performance than as-built members because of previous damage, the stiffness was reduced, retrofitted columns higher normalized shear capacities and deformation capacities were comparable. The existing bars had larger deformation and the bond between the bars and the concrete was reduced considerably prior rehabilitation. However these innovative methods had better response in stiffness, normalized shear capacity and ductility compared with the alternative procedures.

The ultimate drift ratio of the test for retrofitted columns reached values larger than 2% which is the acceptable lateral drift capacity recommended by ASCE07-10 for occupancy category I and II. Retrofitted columns exhibited good performance at 1% drift that is appropriate for occupancy category IV essential infrastructure.

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