PERFORMANCE OF DAMAGED COLUMN RETROFITTED WITH INNOVATIVE MATERIALS AND DEVICES

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SUMMARY:
Natural disasters such as earthquakes and tsunamis may damage or result in collapse of concrete buildings and bridges. Some damaged structures could be restored through retrofit procedures that are cost and time effective. Using innovative materials or devices for strengthening of RC concrete members offers interesting approaches. By assessing the cost to demolish and rebuild a new column with repair of a damaged one; repair may be less expensive than replacement. However, there is very little research regarding the evaluation of structural vulnerability when innovative materials or devices are used. Laboratory tests are being conducted on strengthened elements or substructures. The test program includes the use of CFRP for strengthening or for creating ductile elements. In some tests, mechanical couplers are used to provide continuity to the reinforcement. In location when the rebar has buckled and concrete has crushed, mechanical couplers are used to replace the buckled bars.

Keywords: Retrofit, Carbon Fiber Reinforced Polymer (CFRP), Mechanical Splice

1. INTRODUCTION

Older structures in seismic zones are likely to be damaged in an earthquake. Traditional procedures to repair and strengthen damaged member have generally involved the use of concrete or steel jackets, adding new members to the lateral force-resisting structure, or replacing the damaged element. Several of the studies of such procedures are discussed. The intent of this paper is to outline some new approaches to minimize the need for replacing damaged columns that are being tested at the University of Texas.

2. OBJECTIVE

The most common location of damage to columns occurs where flexural hinges develop. Such sections often suffer considerable loss of concrete in the hinging region and severe distortion of the longitudinal reinforcement. Carbon Fiber Reinforced Polymers (CFRP) are being used to jacket columns that have not been severely damaged. Mechanical couplers are bind used to replace severely bent bars or to replace lap splices that cannot develop the strength of the reinforcing bar. Nonlinear analysis will be used to evaluate the capacity of the repaired elements.

3. BACKGROUND

3.1. Bett, 1985

A reinforced concrete column with a12in x 12 in cross section and reinforced with 8 #6 longitudinal bars and 6mm dia.ties @ 8” was built and test under axial and lateral loads. The ends restrained against rotation.. The column was then repaired using concrete jacketing that increased the section to 17”x 17” by schotcrete with f’c_= 4.6ksi, and adding #6 and and #3 longitudinal bars and #3 ties @

9”to the jacked. The retrofitted column , 1-1R, was tested under the same pattern of load and supports restraints.

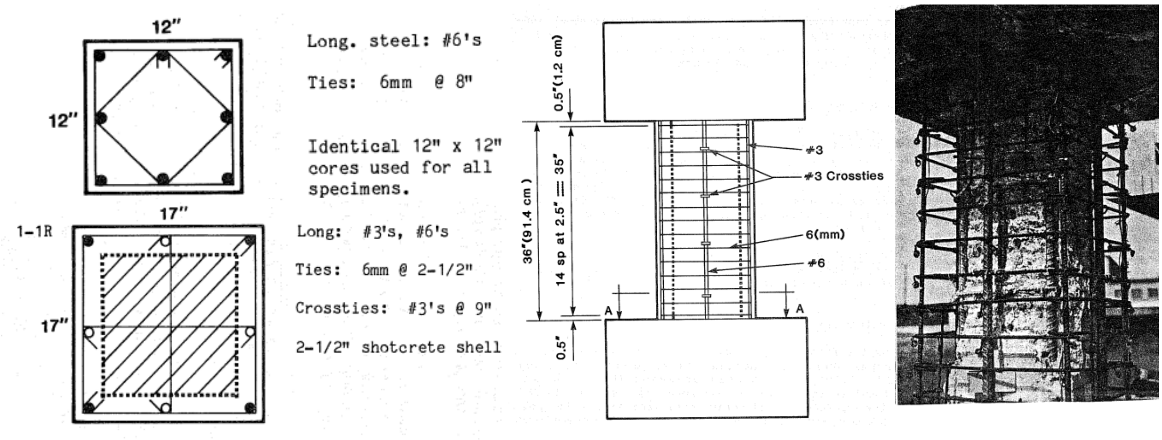


Figure 1. Cross Section of the 1-1R and image of the bars for jacketing (Bett, 1985)

3.2 Aboutaha, 1994

Effects of inadequate lap splice length was studied in a laboratory test program of more than 20 cantilever columns with rectangular and square cross sections. Steel jackets were added to each face of the columns. Column FC-17 had an 18”x18” cross section, and 8 #8 longitudinal bars with #3”@16” for stirrups. The lap splice’s length was 24in. The ¼” thick steel was attached with steel angles at the corners. Two epoxy-grouted steel bolt anchors were added on one face to improve the confinement of the splices on that face.

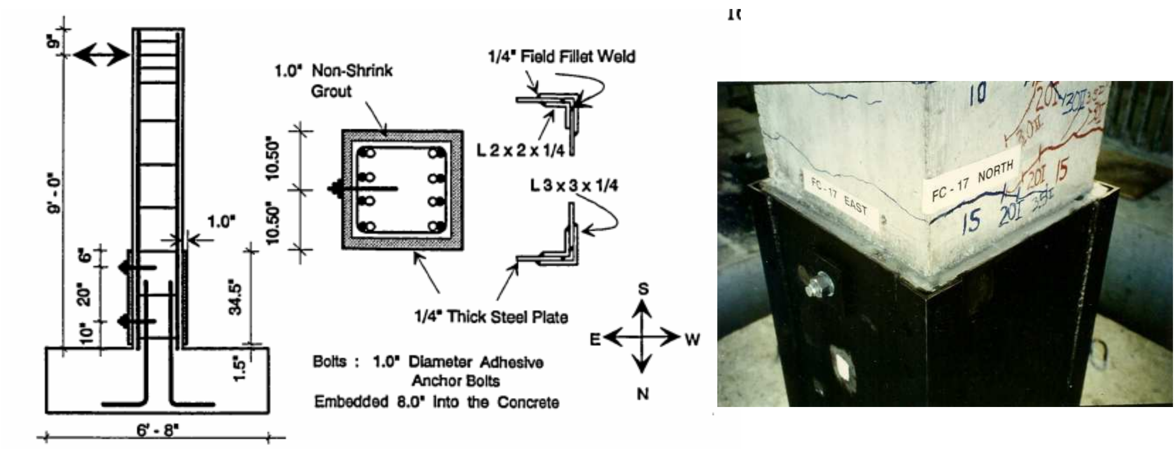


Figure 2. Cross Section of the FC-17 with details of the steel jacket (Aboutaha, 1994)

3.3 Kim, 2008

Similar to Aboutaha’s research, Kim studied the behaviour of columns with poorly detailed lap splices and insufficient confinement. Carbon Fiber Reinforced Polymer CFRP was used for strengthening the splice region. Six cantilever columns were repaired and tested with CFRP jackets and intermediate CFRP anchors. One of the columns, 2-A-S8-M, had an 18in x 18in cross section and 8 longitudinal #8 bars with #3 ties @ 16in. This column was tested under monotonic lateral load and repaired afterwards.

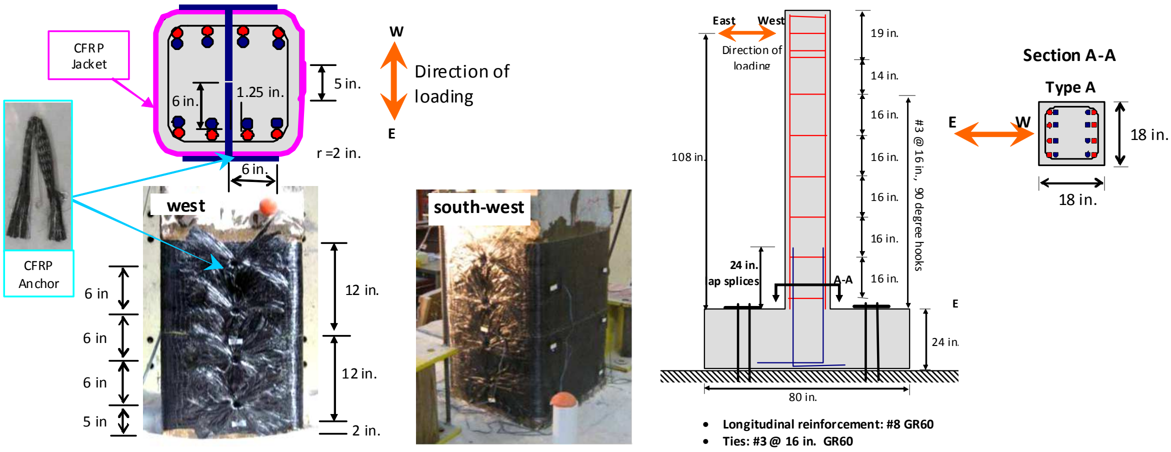


Figure 3. Cross Section of the 2-A-S8-M with details of the CFRP material applied (Kim, 2008)

3. EXPERIMENTAL PROGRAM

3.1. Mechanical Splices

The use of mechanical splices has not been widely used for the rehabilitation of damaged structures. After an event that causes damage to a structure, a CFRP jacket could be used to confine the damaged region provide that the bar are not bent or buckled in the damaged region of columns and walls. If the damage is more severe, the damaged bars may have to be removed. The damaged region of the can be repaired by installing new bars that are mechanically spliced (coupled) to the existing rebar. There are different types of couplers that vary according to the process of the installation.

For this study, the two bars are connected by a sleeve with the bars held tightly in the sleeve with bolts that are torqued to a prescribed level. The system is designed to develop 100% or 125% of nominal yield strength of the bars..Two different mechanical splice configurations were evaluated. For #8 bars one configuration had a number had a 6.8in length and 6 bolts and the other had a10in length and 8 bolts.

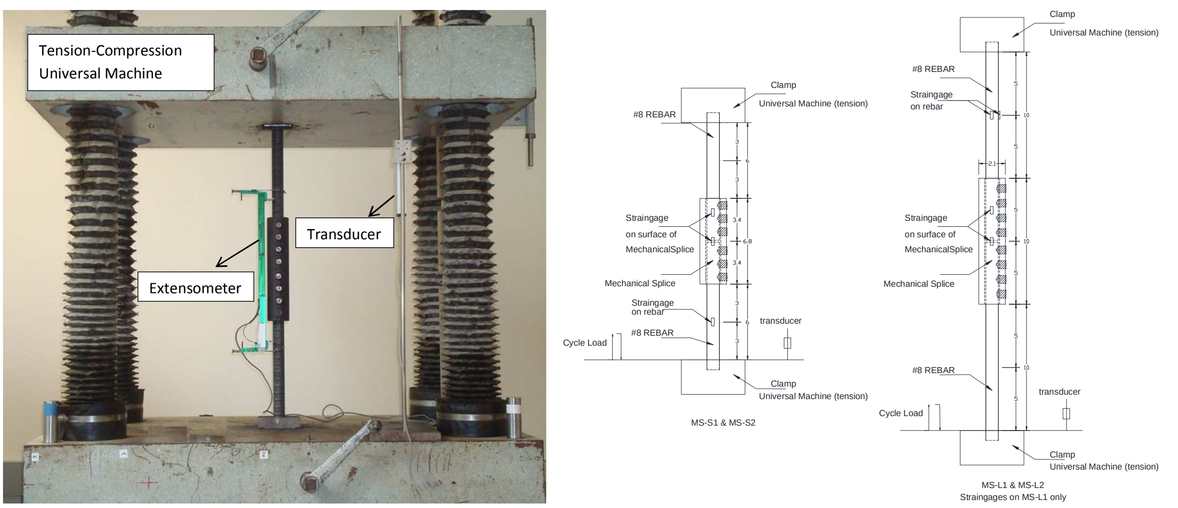


Figure 4. Test Setup for the tension cycle test and details of the mechanical splices tested

The failure of the long splices was fracture of bar at 76.0kips and 75.8 kips. For the short splices the failure mode was bar fracture at last bolt in the sleeve at 68.9 kips and 73.3 kips.

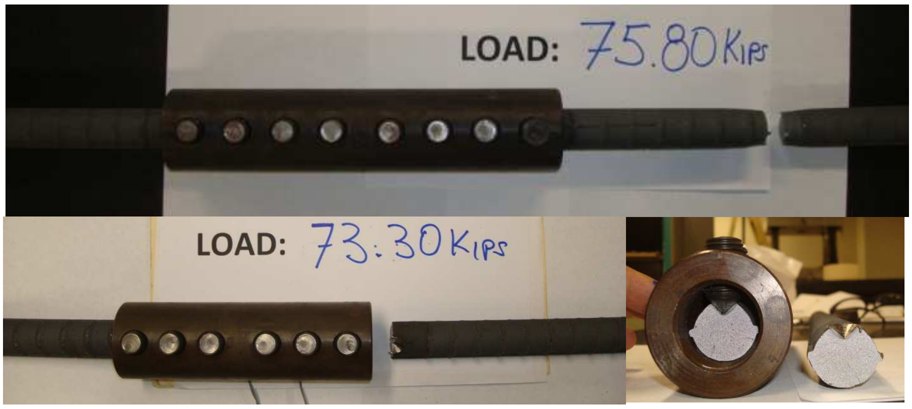


Figure 5. Short Mechanical Splices failure pattern

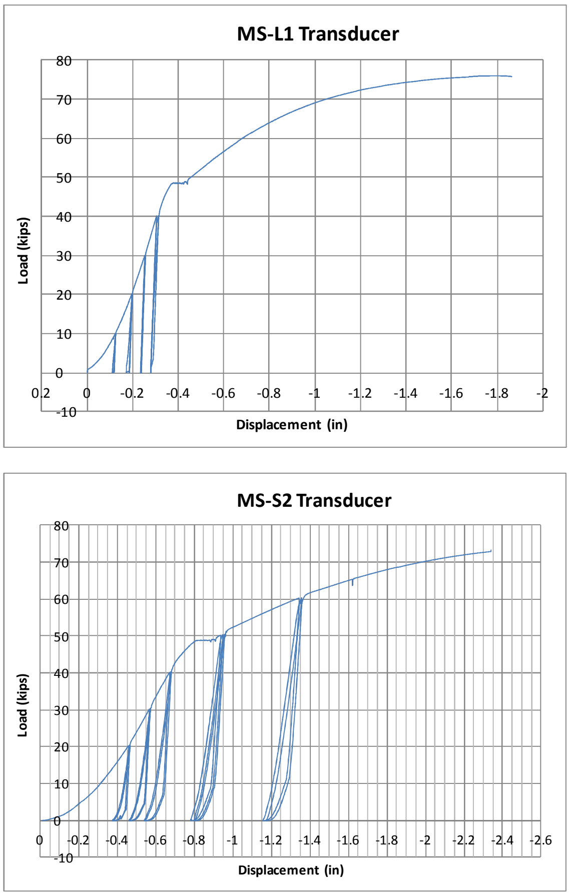
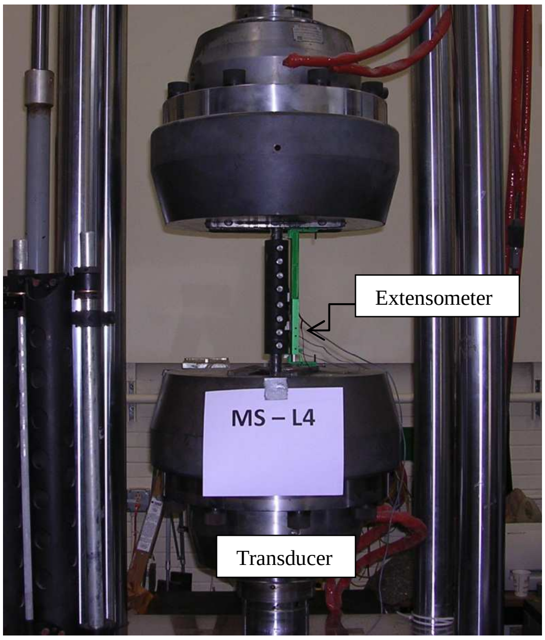


Figure 6. Deformation measured for the system and the mechanical splice MS-L1 and MS-S2

Table 1. Loads measured from the tension cycle test

Specimen	Yield Stress (ksi)	Fracture Stress (ksi)
MS-L1	60.92	96.75
MS-L2	61.36	96.51
MS-S1	62.22	87.83
MS-S2	62.54	93.33

Under cyclic tests, the compression behaviour was similar to that in tension. Those responses induce a good performance of the couplers into the vertical concrete members. Using the coupler test results, a preliminary calculation of the load – deformation capacity under lateral loads of RC square section column was made.



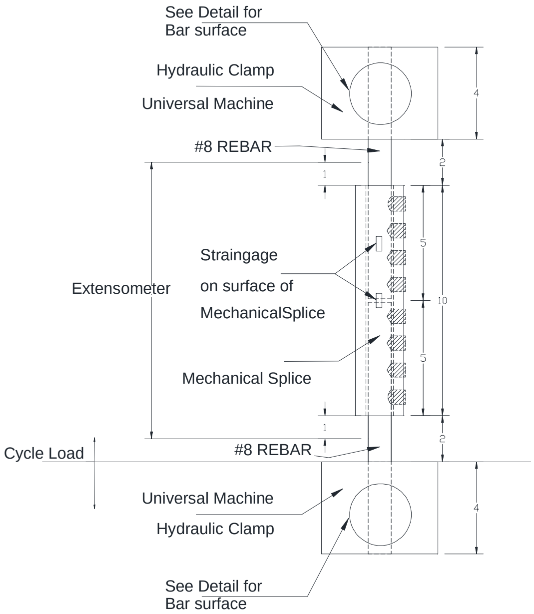


Figure 7. Test Setup for the Tension-Compression Cycle Test Machine and details of the specimens

Failure Mode for the long splices was due to fracture in the contact zone of last bolt (sphere point) with the bar at 85.8kips and 86.0 kips.

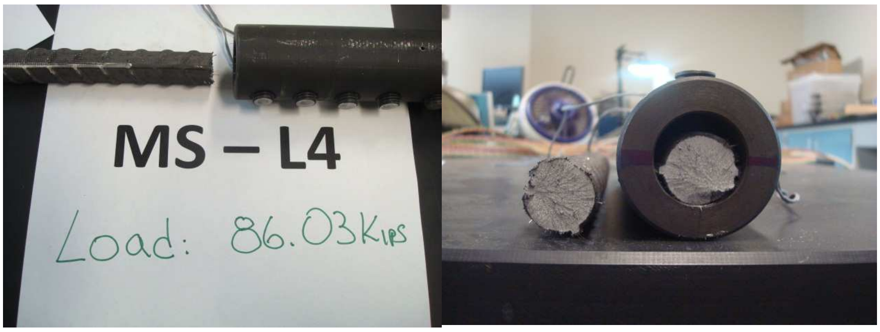


Figure 8. Mechanical Splices failure pattern

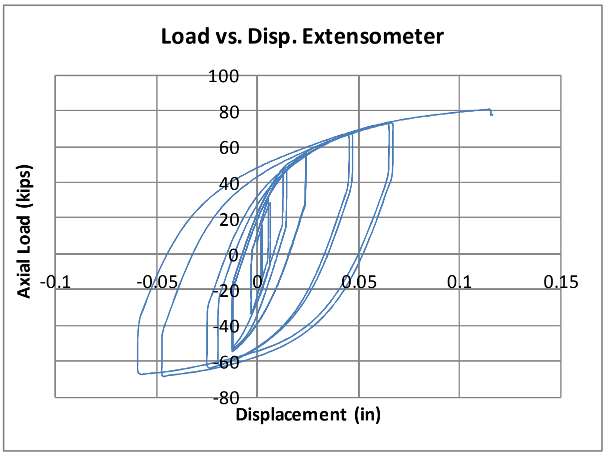


Figure 9. Load-deformation curves for MS-L4

Table 2. Loads measured from the tension – compression cycle test

Specimen	Yield Stress (ksi)	Fracture Stress (ksi)
L3	83.40	109.28
L4	66.65	109.54

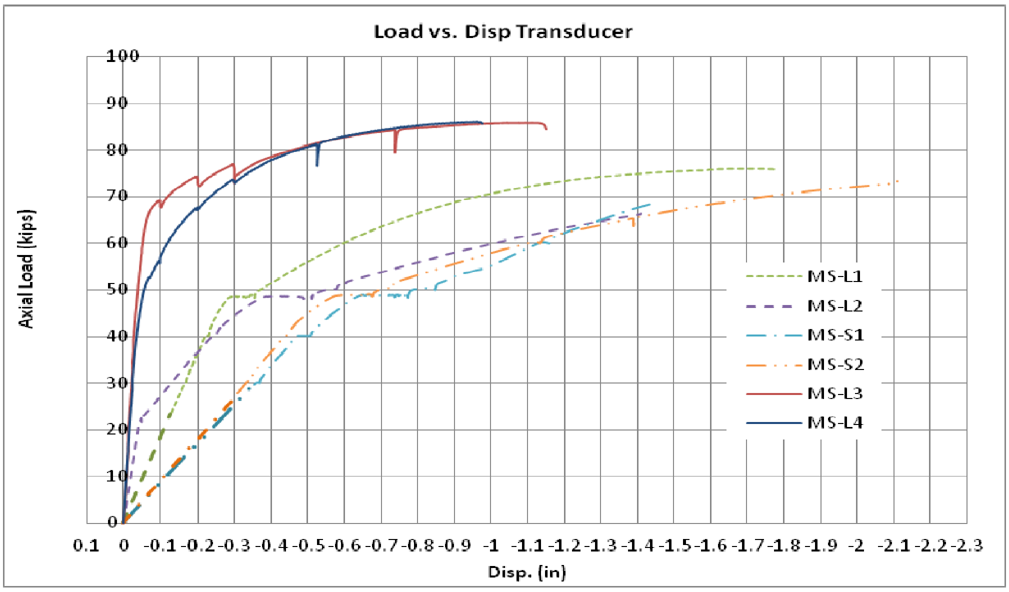


Figure 10. Envelopes Load-deformation curves for the mechanical splices

3.1. Full Scale Tests of RC Columns

3.1.1 Previous Retrofit

Two reinforced concrete columns were built by Leborgne, (2012). Those columns had the same geometry properties. They were tested under lateral cyclic loads and constant axial loads. The columns had 16in x 16in cross section and 8-#8 longitudinal bars with three bars in each face, Column ties were #3 @ 6” with 90 degrees hooks. At each spacing, one perimeter tie was place along with a smaller square tie that confined the middle bars on each side The nominal concrete strength was f’c = 3ksi.

In the test setup, the bottom support of the columns was fixed and at the top, the rotation was restrained to produce a column in double curvature.

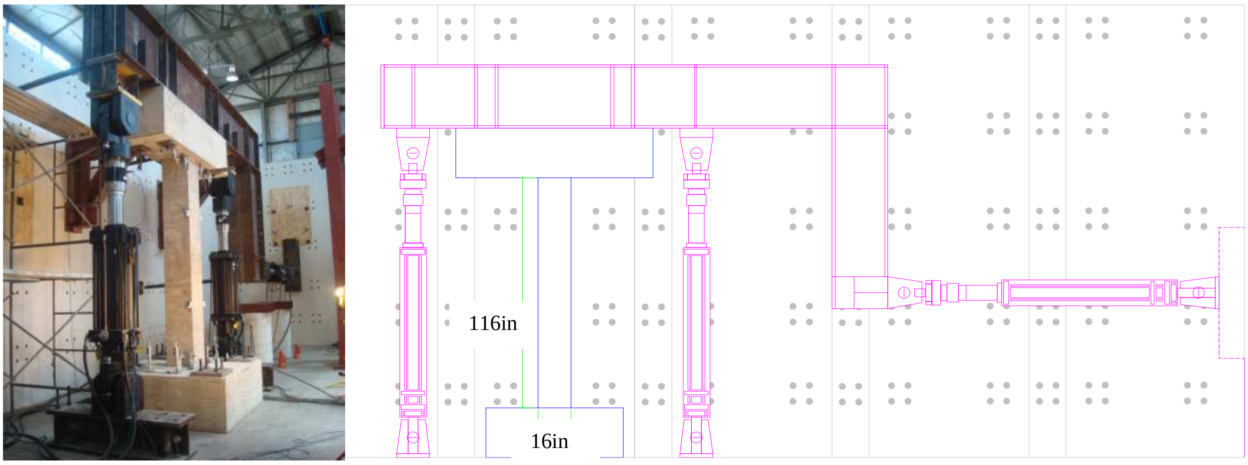


Figure 11. Test setup for the fix bottom – top rotation restrained column.

For the first column DC-150AL: the axial load was 150 kips. Yielding of the longitudinal bar was noted at 0.8in of lateral deformation and bars at both end of column yielded. Shear cracks developed at 15in above the column base at 1.8in of lateral deformation. Finally an axial failure was reached at 6in lateral deformation. 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.

For the test of the second column DC-350AL: the axial load was 350 kips. At 0.76in lateral deformation, yielding of the longitudinal bar at the top of the column was noted. Shear cracks developed at 1.85in of lateral deformation. Axial failure was reached with the column exhibiting severe deterioration at both ends and buckling of longitudinal bars coupled with crushing of the concrete.

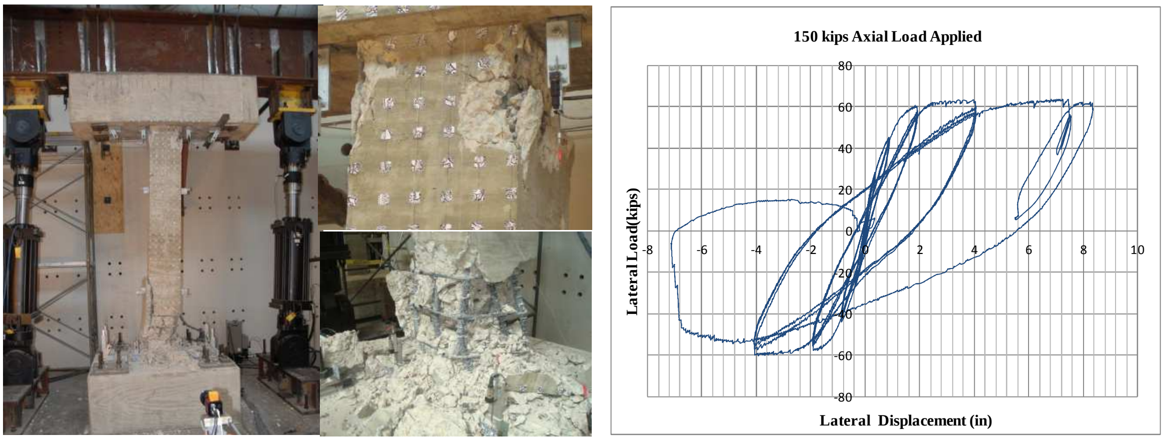


Figure 12. First column test and lateral load – deformation curve

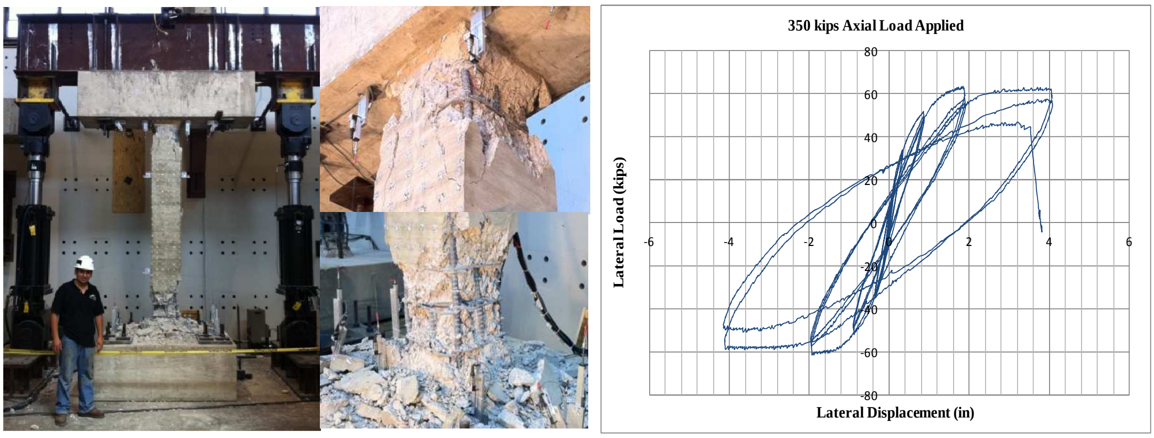


Figure 13. Second column test and lateral load – deformation curve

3.2 The retrofit of the column damaged.

For DC-150AL 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 concrete removed. The damaged regions were wrapped with a CFRP sheet and some intermediate CFRP anchors were installed midway between the corners. At the bottom of the column, short mechanical splices were use to replace the buckled bars and to provide continuity to the longitudinal bars.

For DC-350AL, the 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. One of the half-columns was repaired using long mechanical splices described above and the other using the shorter couplers.

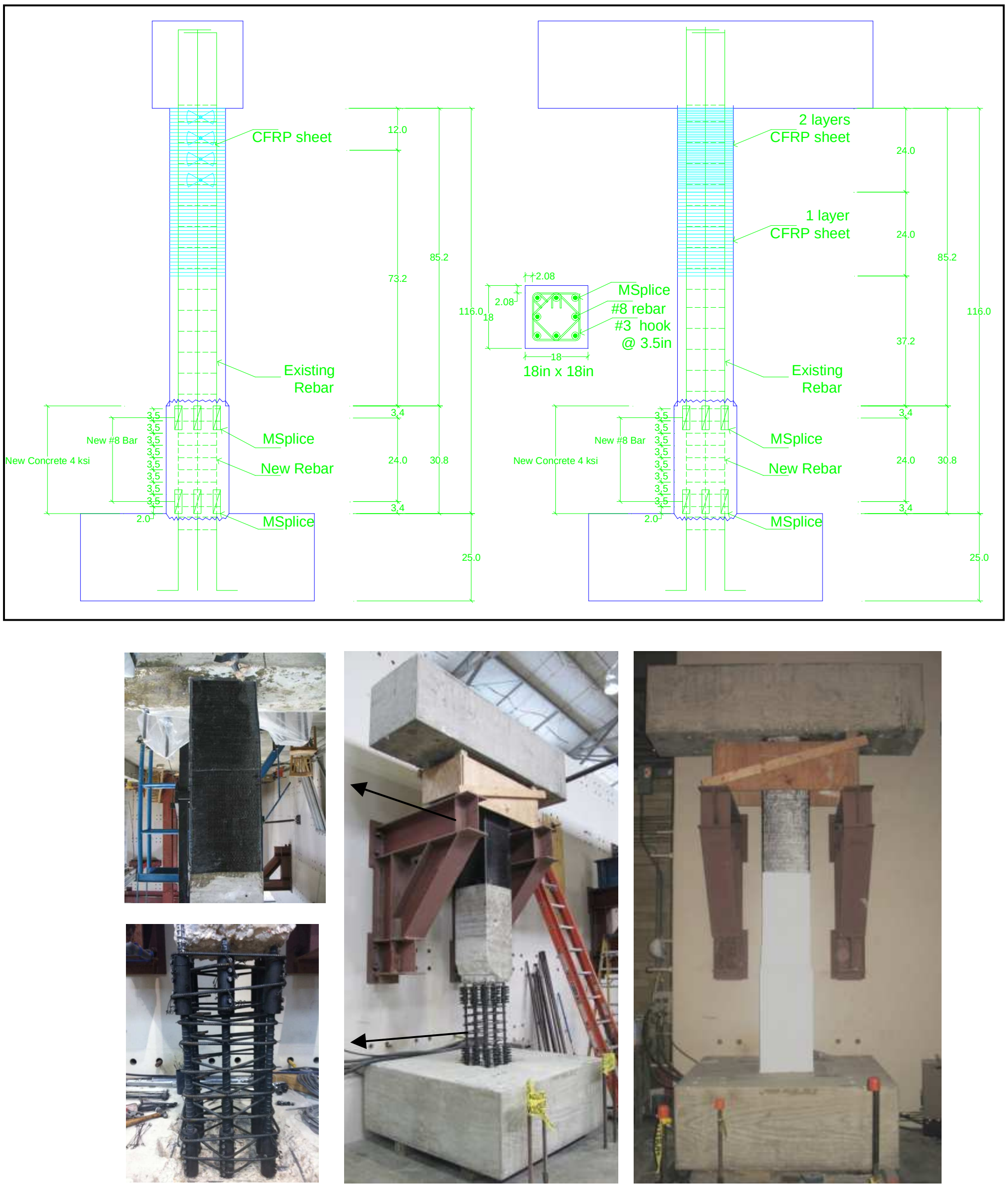


Figure 14. Retrofit of the first column

Figure 15. Retrofit of the second column divided in two new cantilever ones (Left pictures : MS-S and Right pictures: MS-L column)

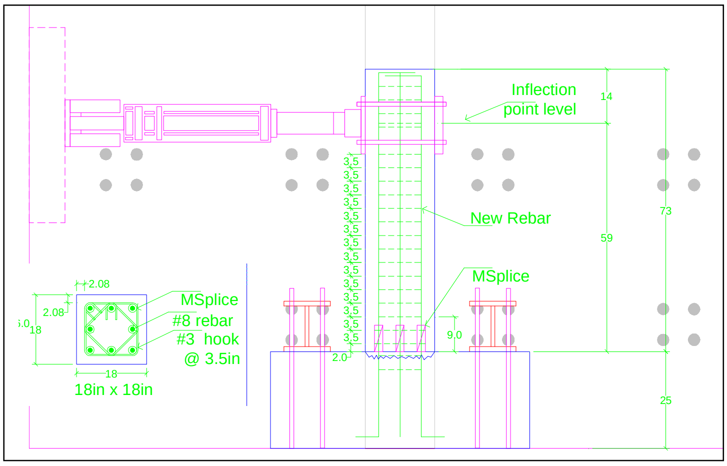


Figure 16. Setup for the test of the two new cantilever columns

4. DISCUSSION AND COMPARISSON OF RESULTS OF PREVIOUS TESTS AND PRELIMINARY PREDICTION OF BEHAVIOR FOR RETROFITTED PROPOSED CASES

A preliminary calculation of the capacity of a cantilever reinforced concrete section under lateral load was performed in order to assess numerically the capacity of the cantilever column cases from DC-350AL. The geometry of the retrofitted columns, the concrete strength and the measured material properties of the long and short mechanical splices were used in the analysis. For concrete, Scott, Park & Priestley material curve was considered.

The load and drift capacity from the two cantilever columns (MS-S & MS-L) have been compared with the columns tests reported by Bett (1-1R), Aboutaha (FC-17) and Kim (2-A-S8-M) cases, and with the original columns tested (DC-150AL & DC-350AL). An extra case was assessed, #8, which represents a column with the same cross section geometry and concrete quality of MS-S and MS-L, however with #8 bars instead of the mechanical splices. All of those cases were normalized respect to the nominal strength design load for each section. It should be noted that the strength of the repaired columns has been restored and the ductility significantly improved relative to the original members.

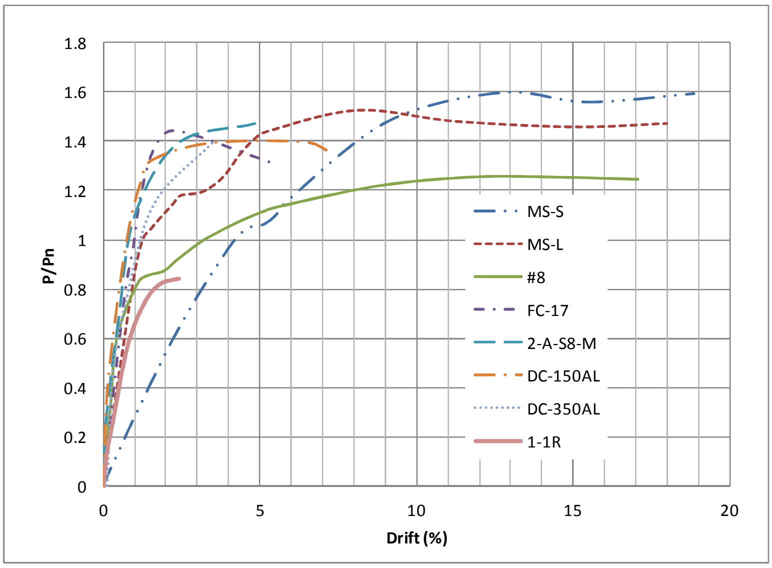


Figure 17. Mathematical models for non-linear response history analysis compared with previous research cases.

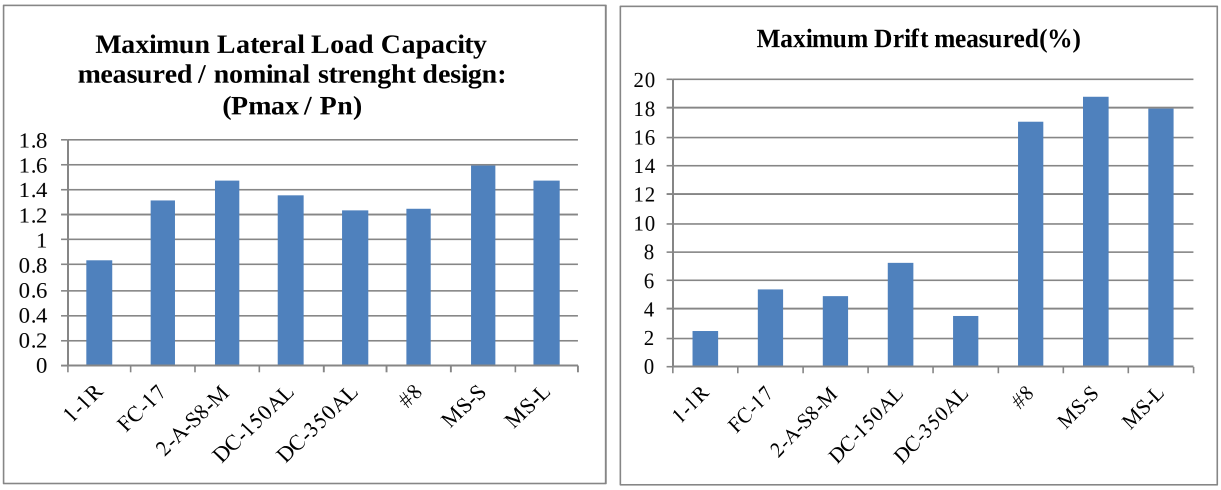


Figure 18. Preliminary predictions for response history analysis compared with previous research cases

5. CONCLUSION

The research in progress indicates that the use innovative materials and retrofit techniques result in performance that is likely to be equal to or better than that using more conventional techniques. The feasibility of using such techniques depends on the degree of damage, the cost of replacement, and the performance required.

AKCNOWLEDGEMENT
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