Structural Upgrade of Reinforced Concrete Column-Tie Beam Assembly using FRP Composites

by A.S. Mosallam

Synopsis: The paper discusses the potential use of fiber reinforced polymer composites for repair and retrofit of existing reinforced concrete (RC) column-tie beam assemblies. Results of an experimental program performed on large-scale specimens repaired and strengthened with two types of wet lay-up composite systems are presented. Each column-tie beam assembly specimen was subjected to a constant axial load simulating gravity loads, and incremental cyclic lateral loads simulating potential seismic forces. Displacements, strains and loads were continuously monitored and recorded during all tests. Evaluations of the observed strength and ductility enhancements of the strengthened specimens are made and limitations of such retrofit methods are highlighted for design purposes. Experimental results indicated that the two composite systems used in this study succeeded in enhancing the strength, stiffness and the ductility of the column-tie beam assembly. As compared to the unstrengthened specimens, the strengths of the retrofitted specimens were 152% and 154% for carbon/epoxy and E-glass/epoxy composite systems, respectively.

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INTRODUCTION

Over the years, several seismic repair and retrofitting techniques, including pressure epoxy injection, epoxy impregnation [1], external steel plating [2], and concrete jacketing [3] have been investigated. However, there are a number of drawbacks associated with these techniques. The 1995 Federal Emergency Management Agency (FEMA)-97 Report questions the effectiveness of the pressure injection technique in completely restoring the bond between the reinforcement and concrete. The epoxy impregnation method, while able to overcome the problem of the creation of voids in small-scale tests, is extremely difficult to apply on large practical scales. The use of bonded steel plates has several disadvantages, including high cost, possibility of corrosion of the steel at the bond-line interface (which can lead to premature bond failure), and the requirement for specialized heavy equipment at the job site.

As compared to steel jacketing, the use of polymer composites repair and retrofit systems provide several unique advantages.   The key advantage of composites in these applications is its tailorability, which enables the engineer to decouple stiffness and strength (stiffer columns would attract more forces during the earthquake which are not included in the original design). In the last decade or so, the use of fiber reinforced polymer (FRP) in repair and rehabilitation of different reinforced concrete members has been widely accepted by the structural engineering community. Some of the early successful application of FRP composites was in the repair and retrofit of reinforced concrete highway bridge columns that started in California and now is considered to be standard accepted procedures by different departments of transportation in USA and other countries. This application was extended to buildings and other constructed facilities where hundreds or may be thousands of repair and rehabilitation projects have been successfully executed.   The use of these materials was shown to be also effective in repair and rehabilitation of reinforced concrete girders, slabs, shear walls as well as masonry wall structures [4, 5].

One of the recent applications of composites is repair and rehabilitations of different types of reinforced concrete joints including shear capacity upgrade of beam-column joints [6, 7] moment frame connections and column-tie connections [8, 9] as described in this paper. The major influence of connections details on the structural integrity and seismic performance of reinforced concrete structures has more evident after the 1989 Loma Prieta, the 1994 Northridge and 1999 Kocaeli (Turkey) earthquakes. Post earthquake reports of the Loma Prieta indicated that one of the main reasons behind the collapse of the Cypress Viaduct, and the damage of the China Basin and the I-80 Freeway is the failure of connections. As the result of 1994 Northridge earthquake, several parking structures collapsed mainly due to severe damage of beam-column and column-base connections (see Figure 1). During more recent events such as the 1999 Kocaeli (Turkey) earthquake, it was observed that beam-column joint failures contributed to the severe damage and collapse of many reinforced concrete buildings [10] (see Figure 2).

Column-tie connections are found in many structures including space frames that are commonly used in oil refineries and petrochemical facilities (see Figure 3) and some parts of moment frame buildings (see Figure 4). For this special connections, no gravity loads are applied to the tie beam (no floor slabs), except of its own weight, and all vertical loads are transferred via columns as shown in Figures 3 and 4.   In this case, the joint is exposed to minimum shear and relatively higher moments and axial loads in the column region. In this paper, a pilot study on repair and rehabilitation of this type of joint using two types of FRP composites is presented.

EXPERIMENTAL PROGRAM

In this study, two composite strengthening systems for column-tie beam connections were evaluated, that included i) carbon/epoxy wet layup system and, ii) E-glass/epoxy wet layup system. Table (1) presents the average values of the mechanical properties of the two FRP composites systems.   The values presented in Table (1) were obtained from coupon specimens that were tested in accordance to ASTM D-3039 "Standard Test Method for Tensile Properties of Fiber-Resin Composites".

Loading History

The loading regimes used in all evaluation tests described herein adhered to the ICC AC125 requirements as shown in Figure (5). In this loading scenario, the effect of seismic forces was simulated by applying reversed cyclic loading to the tip of the beam member while maintaining a constant axial load to the column. Per the ICC-AC 125 requirement, the lateral reversal load and displacement histories were divided into two phases. Initially, the tests were conducted under a load control mode until the yielding load of the connection' steel reinforcements. At this stage, a displacement control mode was utilized.

The test setup was designed such that the specimens are subjected to constant axial load and cyclic vertical loads. An axial load was applied to each horizontal member just before the test. The peak forces controlled the initial loading cycles until the specimen developed the force corresponding to the first yield of longitudinal steel, V_y. Then, the test was stopped and the yield displacement was calculated from the following equation:

$$\Delta_y = \frac{V_i}{V_y}\Delta_1 \tag{1}$$

where Δ_1 is the average of the measured peak displacements corresponding to the first-yield lateral load, V_y, in the push and pull directions. The ideal flexural lateral load capacity, V_i, is computed based on the extreme concrete compressive strain of 0.004 (*0.005 for retrofitted specimens*) and on measured material properties. After the specimen developed the first yield capacity, loading cycles were controlled by the peak displacement till failure.

Displacement Ductility

Displacement ductility factor μ_Δ, was used in defining the loading history. The ductility factor is defined as the ratio of the applied displacement (Δ) over the displacement at first yielding of the connection' steel reinforcements *(Δ_y)*. The yield displacement is defined by the following equation:

$$\Delta_y = \frac{M_{if}}{M_y}\Delta_1 \tag{2}$$

where Δ_1 is the average measured displacement corresponding to the first yielding moment capacity, M_y, in the push/pull direction.   The ideal flexural moment capacity, M_{if}, is calculated based on the concrete compressive strain and the measured materials properties.

Test Specimens

A total of four full-scale reinforced concrete column-tie beam assemblies tested under sustained axial column load and full lateral reversed cyclic loading conditions. The specimen represents a typical construction similar to those shown in Figures (3) and (4). The ends of the columns were pinned simulating inflection points. The column-tie beam assemblies were configured to induce moment limit-states and related failure modes. As shown in Figure (6) extremes of dimensions and weak reinforcement details were employed per the recommendations of section 5.3.1 of the ICC-ES AC 125 [11].

Specimens' Lay-up

As it was mentioned earlier, two lay-up schedules were used for the carbon/epoxy and E-glass/epoxy systems. The general fiber architecture for both systems was similar and the only difference was in the number of plies in some directions. The general lamination concept is to strengthen the joint portion with quasi-isotropic laminate [0°/90°/±45°] and the beam and the column portion with cross-ply laminate [0°/90°]. However, wrapping the 90°-laminate was only applied to the column portion above and below the floor level (beam's upper and lower faces) since wrapping the beam (which is desirable) is not practical in building applications because of the presence of the floor slab. To account for possible obstruction of the slab that will exist in an actual building, unstrengthened depth of 4" (101.6 mm), simulating the depth of the floor slab was maintained.

Control (Unstrengthened) Moment Frame Joints Tests

In order to develop a baseline for comparing and quantifying the contributions of the composite strengthening systems, two identical control (*unstrengthened*) joint specimens were tested. Figure (6) shows the dimensions and reinforcement details for an unstrengthened interior column-tie beam joint (refer to Figures 3 and 4). The typical test setup for all specimens is shown in Figure (7).

In performing this test, the loading history shown in Figure (5) was employed. Initially, load-control regime was adopted until the first yield, after which displacement-control loading was used. Figure (8) shows the control specimens during testing and the progression of failure. The load-deflection hysteresis curve for one of the unstrengthened control specimen is shown in Figure (9). The failure of the control specimen was brittle with an average ultimate lateral load of 19.55 kips that is corresponds to a moment capacity of 36.90 kip-ft (50 kN-m) and a maximum rotational angle of 0.03 radiant. The average idealized yield displacement of the control specimens was 0.34" (8.64 mm). Table (2) presents a summary for the average experimental values obtained from the control joints tests.

Carbon/Epoxy Moment Frame Joints Tests

Initially, the connection exhibited high stiffness which lasted for the first four cycles and up to a lateral displacement of about 1" (25.4 mm). At this stage, the joint reached its maximum load capacity of 29.85 kips (132.8 kN) which corresponds to a maximum flexural capacity of 55.10 kip-ft (74.71 kN-m). In the following cycles, gradual degradation in both stiffness and strength was noticed as shown in Figure (10). The idealized yield displacement was 0.67" (17 mm) and the ultimate displacement was 2.23" (56.64 mm). The joint relative rotation measured using the four LVDT's at the ultimate load was 0.041 rad. The maximum calculated ductility of this specimen was 3.6.

Test results indicated that the flexural strength of the column-tie beam assemblies strengthened with the carbon/epoxy system was 1.53 times the average strength of the unstrengthened "as-built" specimen (29.85 kips (133 kN) vs. 19.55 kips (87 kN)). The ultimate lateral displacement at failure for the strengthened specimen was 1.24 times the corresponding displacement of the unstrengthened specimen (2.32" (59 mm) vs. 1.87" (47.5 mm)). In addition, test results indicated that the use of the carbon/epoxy FRP system resulted in an appreciable stiffness enhancement to the strengthened specimen as compared to the control one. The initial rotational stiffness has been increased to 15,000 kip-ft/rad (20,317 kN-m/rad) as compared to only 4,175 kip-ft/rad (5,655 kN-m/rad) for the control joint specimen with gain in the initial rotational stiffness of 260%.

The ultimate failure of this specimen was a combination of spalling of the unstrengthened concrete portion of the joint that was extended to the top side of the tie beam (refer to Figure 11). This local joint damage resulted in a large joint rotation followed by a rupture of column's steel rebars at joint location as shown in Figure (11). The maximum-recorded FRP laminate strain was 0.68% which is about 60% of the rupture strain of the carbon/epoxy composite system.

E-Glass/Epoxy Column-Tie Beam Joints Tests

Similar to the previous evaluation test, the objective of this test was to demonstrate the structural performance and the effectiveness of the E-glass/epoxy composite system in enhancing the cyclic performance of a deficient reinforced concrete moment frame joints. The FRP lamination schedule for this specimen was similar to the carbon/epoxy strengthened specimens described earlier. After applying the composites, the composites material was allowed to cure for at least 72 hours under the laboratory environment. Strain gages at different critical locations and in different fiber directions were bonded to the external composite laminate as well as to the internal steel rebars. Figure (12) shows the $P\text{-}\delta$ hysteresis loops for the E-glass/epoxy composite strengthened joint specimen. Similar to the carbon/epoxy joint specimen, the joint exhibited a relatively high stiffness which lasted for the first five load levels (fifteen cycles), after which a gradual degradation in both stiffness and strength was observed. At this stage, the joint reached its maximum average load capacity of 32.7 kips (145.5 kN) which corresponds to a maximum flexural capacity of 60.36 kip-ft (81.51 kN-m). As shown in Figure (12), the strength degradation following the fifth cycle was very moderate (*only 9% drop*). The idealized yield displacement for this specimen was

0.65" (16.51 mm) and the ultimate displacement was 2.52" (64 mm).   The joint relative rotation measured using the four LVDT's at the ultimate load was 0.045 rad. As shown in Figure (13), the ultimate ductility of this specimen was 3.9. Damage progression and ultimate failure are illustrated in Figure (14).   The maximum laminate strain measured during this test was 2.15% which is about 95% of the rupture strain of the E-glass/epoxy composite system.

Test results indicated that the strength of the column-tie beam joint assembly strengthened with E-glass/epoxy composite system is 1.54 times the average flexural strength of the unstrengthened "control" specimen (30.10 kips/133.9 kN vs. 19.55 kips/87 kN). The ultimate lateral displacement at failure for the strengthened specimen was 1.34 times the corresponding displacement of the unstrengthened specimen (2.52" (64 mm) vs. 1.87" (47.5 mm)).

SUMMARY & CONCLUSIONS

Experimental results indicated that the two FRP composite systems succeeded in enhancing the strength, stiffness and the ductility of the seismically-deficient reinforced concrete column-tie beam connections.   As compared to the unstrengthened joint specimens, the strengths of the strengthened specimens were 152% and 154% for the carbon/epoxy and E-glass/epoxy composite systems, respectively.   Figure (14) presents a graphical comparison between the strength capacity of the strengthened and the average strength capacity of the as-built (*control*) joint specimens.   In addition, test results indicated that the use of the FRP composite strengthening systems resulted in an appreciable increase in the maximum lateral (*floor*) displacement at the cyclic ultimate load (20% increase for the carbon/epoxy and 35% for the E-glass/epoxy)*.* Figure (15) presents the load-displacement envelopes for strengthened and unstrengthened (*control*) test specimens. As shown in this figure, both the strength and stiffness of the strengthened specimens were upgraded.   This figure also indicates that the FRP strengthened specimens have a lower rate of strength deterioration and higher ductility up to failure (*increased up to 35%)* as well as higher initial stiffness as compared to the as-built specimen.

ACKNOWLEDGEMENT

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Table 1— Properties of the Composites Materials

System	*Thickness (t) inch [mm]*	*Ultimate Strength ksi [MPa]*	*Strain at Ultimate (μ-strain)*	*Modulus of Elasticity ksi [GPa]*
Carbon/epoxy	0.045 [1.14]	154 [1,061]	0.012	14 X10^3 [96.5]
E-glass/epoxy	0.045 [1.14]	74 [510]	0.022	3.5 X 10^3 [24.2]

Table 2— Average Experimental Values for the two Control Specimens

Ultimate Load, kips (kN)	Average Moment Capacity, kips-in (kN-m)	Average Lateral Displacement, in (mm)	Average Relative Rotation, rad	Rotational Stiffness, kip-in/rad (kN-m/rad)
19.55 (87)	443 (50)	1.7 (43.18)	0.03	50,100 (5,655)

Figure 1— Local Failure of Beam-Column Connections
during Northridge Earthquake, California, 1994. [8]

Figure 2— Building Collapse Due To Failure of RC Beam-Column Joints during 1999 Kocaeli Earthquake. [10]
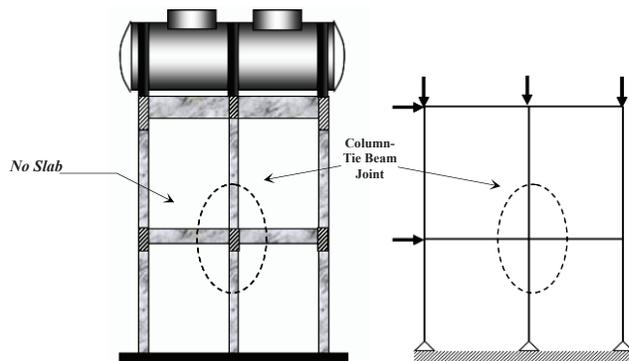
Figure 3— Condenser and Elevated Tank Frame in Oil Refineries.
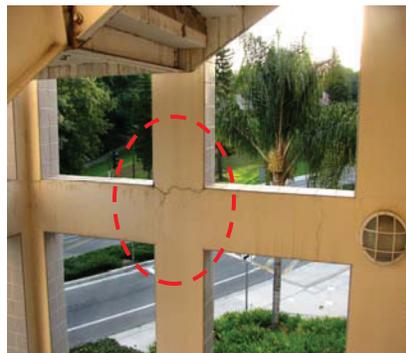
Figure 4— Typical Column-Tie Beam Joint in RC Buildings.
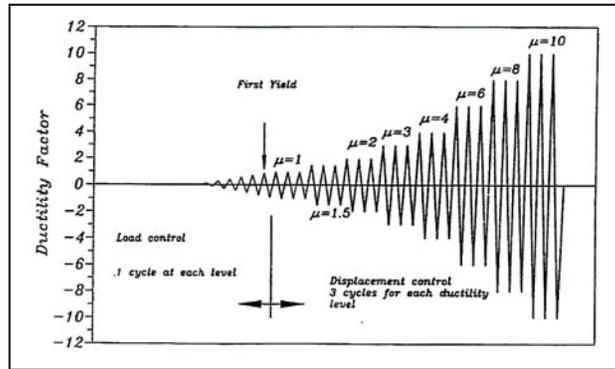
Figure 5— Typical Lateral Loading History.
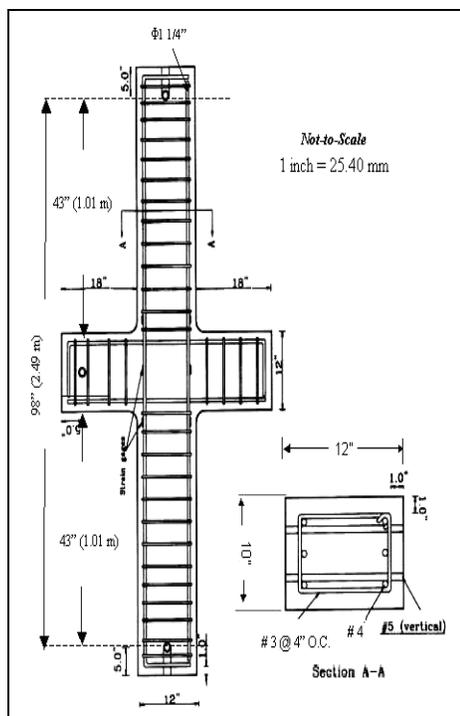*[Source: Fig. 1 – ICC-ES AC 125]*

Figure 6—Dimensions and Reinforcement Details for Interior Control (*Unstrengthened*) Moment Frame Joints.
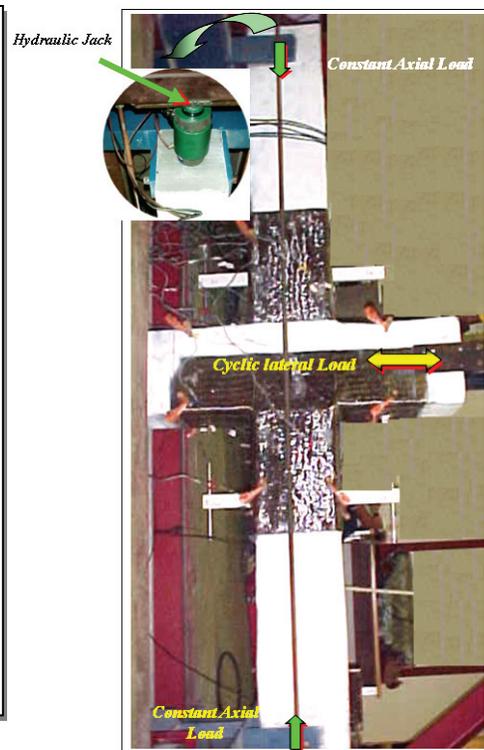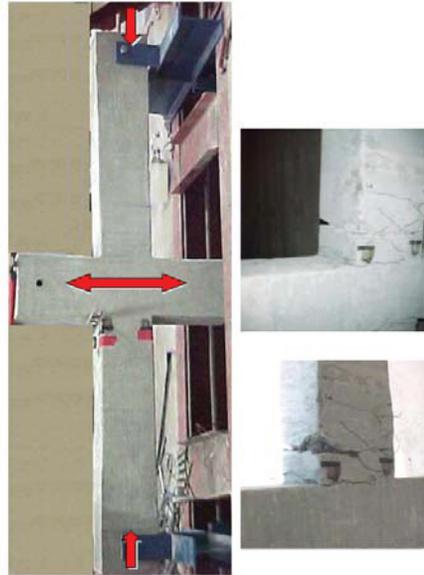

Figure 7—Typical Test Setup.

Figure 8—Control Column-Tie Beam Assemblies Test
Setup and Progression of Failure.
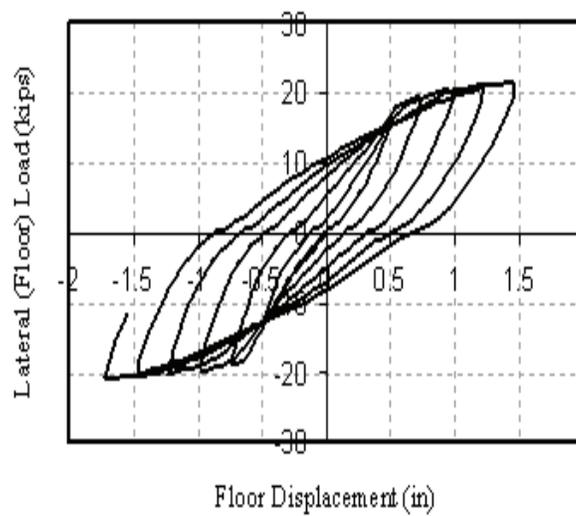
Figure 9— Experimental Load-deflection Hysteresis
for Unstrengthened Specimens.
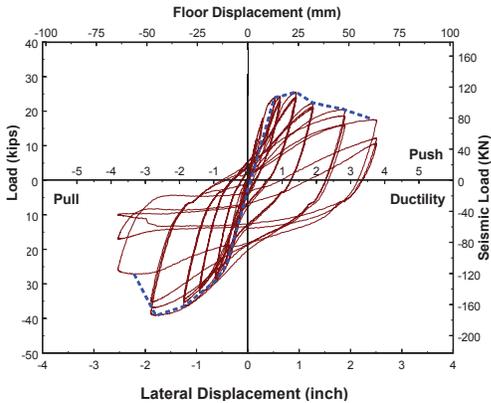
Figure 10— Experimental Load-deflection Hysteresis for
Carbon/Epoxy Retrofitted Specimens.

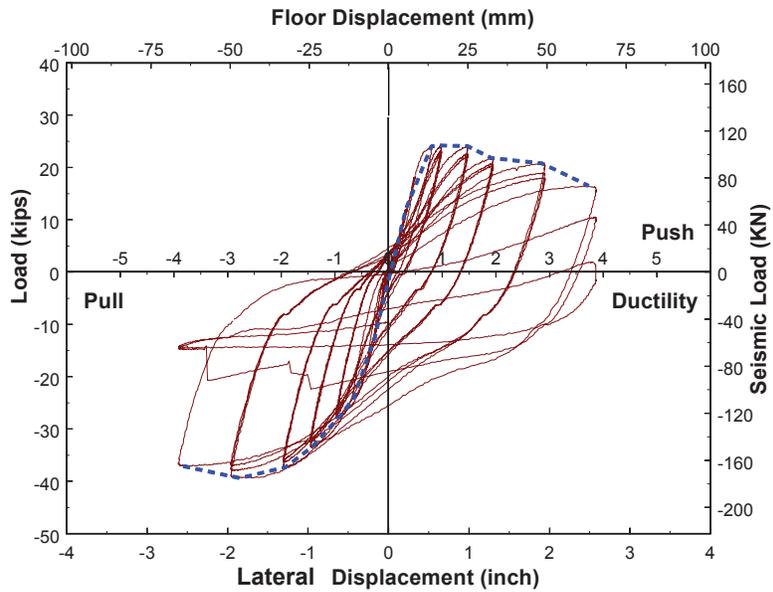**Figure 11—** Ultimate Failure Mode of Carbon/Epoxy
Retrofitted Specimen.

Figure 12— *P-δ* Hysteresis Loops for E-Glass/Epoxy
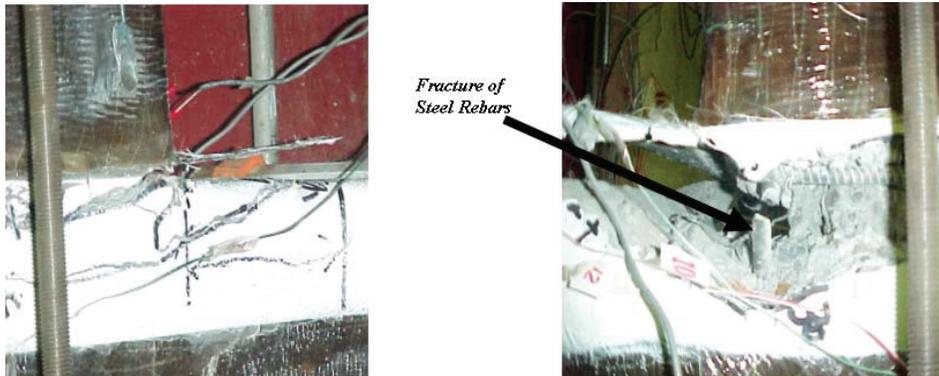Retrofitted Specimen.

Figure 13— Damage Progression and Failure Mode of the E-Glass/Epoxy
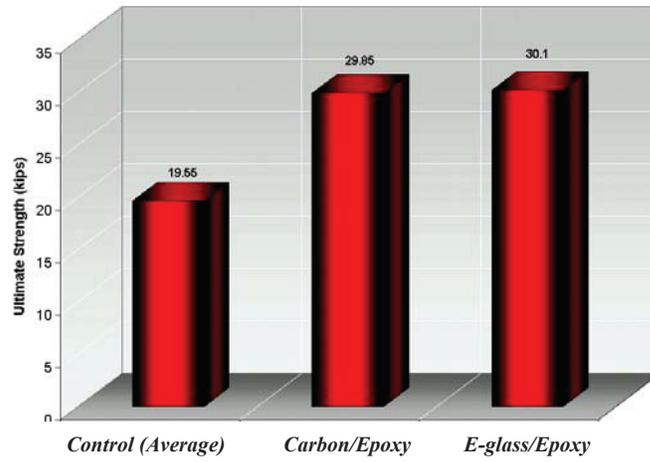Retrofitted Specimen.

Figure 14—Strength Comparison between Strengthened and Unstrengthened (*As-Built*) Column-Tie Beam Joint Specimens.
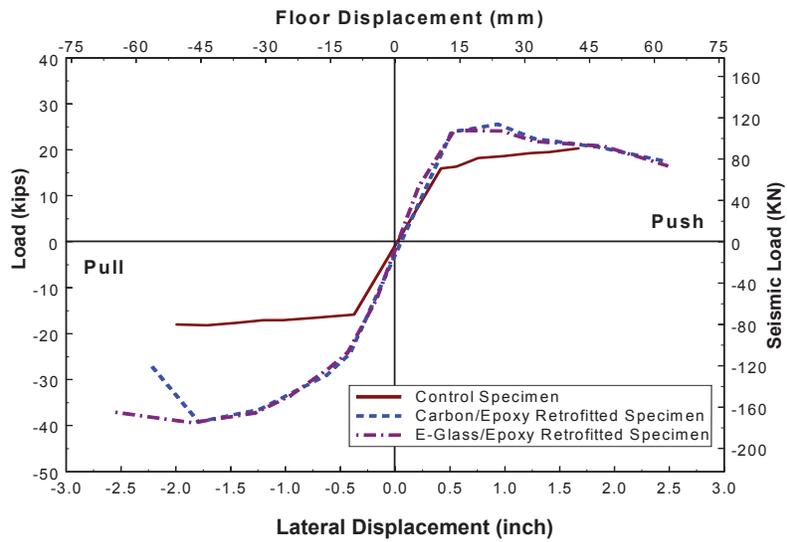

Figure 15— Comparison between Load-Displacement Envelopes for Strengthened and Unstrengthened (*Control*) Test Specimens.