REHABILITATION OF R/C BUILDING JOINTS WITH FRP COMPOSITES

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SUMMARY

Reinforced concrete buildings that were built in the 1960's do not meet current design criteria and behave in a non-ductile manner. In this paper, beam-column joints of such non-ductile buildings are investigated. Half-scale R/C corner joints were tested for the purpose of investigating their behavior in a shear type of failure due to diagonal tension. In addition to the as-is specimens, an identical corner joint was retrofitted with FRP composites to determine the improvement in ductility and joint shear capacity that could be achieved. The proposed method of strengthening the beam-column joint with FRP composite jackets is relatively new; existing techniques include R/C jackets, glued steel plates and X-shaped prestressed collars. The design of the FRP composite jacket for retrofitting the joints to sustain the diagonal tension is described. Both the as-is and retrofitted corner joints were subjected to quasi-static cyclic loading, and their performance is examined in terms of peak lateral load capacity, ductility, drift, axial load bearing capacity of the column at high levels of drift, and in terms of crack widths. The influence of the axial load applied to the column on the joint shear capacity is investigated and compared to suggested values in the literature. The as-is joint reached 66% of the ACI 352R-91 Type 2 joint shear strength. The behavior of the FRP composite retrofitted joint is significantly improved in terms of lateral load capacity, ductility and axial load bearing capacity at high levels of drift. In addition, the joint shear strength of the FRP retrofitted joint was 45% higher than that of the as-is joint.

INTRODUCTION

Reinforced concrete frames can achieve ductile behavior provided that brittle failure of structural elements and instability can be prevented in severe earthquakes. The design and detailing of beam-column joints is important in achieving satisfactory performance of R/C frames. The design should be able to: (a) prevent brittle shear failure of the joint, (b) maintain integrity of the joint so that the ultimate strength of the connecting beams and columns can be developed, and (c) reduce joint stiffness degradation by minimizing cracking of the joint concrete and by preventing the loss of bond between the concrete and longitudinal beam and column reinforcement. Joints in existing structures built prior to the development of current design guidelines such as ACI 352R-91 [1991] do not conform to the current requirements [ACI SP-123 1991]. This research targets the performance of corner joints in existing R/C frame structures in order to establish their adequacy. In addition, the possible improvement in the joint strength and ductility is investigated for a joint retrofitted with FRP composites.

A building located in Los Angeles, California which has experienced several earthquakes including San Fernanado in 1971 and Northridge in 1994 was selected as a model for this study. This reinforced concrete building was built in 1964 and sustained substantial damage due to seismic action. As is typical of buildings built in the early 1960's, the beam-to-column connections lack confining reinforcement in the joint and have insufficient anchorage of reinforcement extending into the connection. Half-scale specimens of a typical exterior joint in the model building were constructed. One as-is specimen and one retrofitted with carbon Fiber Reinforced Polymer (FRP) composite have been tested. The objectives of the as-is and retrofit specimen testing are to: (a) assess the performance of a typical exterior R/C building joint; and (b) determine the improvement in

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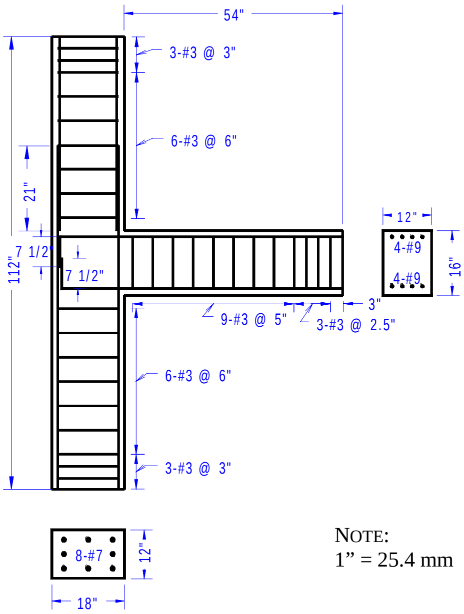
strength, ductility, and drift capacity of an exterior joint with FRP composite retrofit. While all components of the specimen performance were evaluated, the main focus of the study was shear in the joint region.

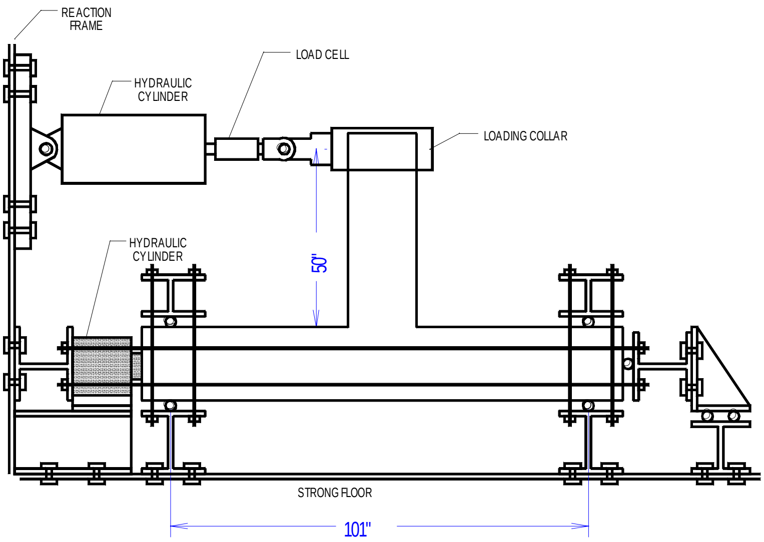
AS-IS SPECIMEN

The overall dimensions of the original exterior joints were reduced by half. Reinforcing details were reduced based on shear stress calculations and then slightly modified to accommodate the desired goals of the research. The specimen dimensions and reinforcement are shown in Figure 1. There is no transverse reinforcement within the joint core, and the beam longitudinal bars are not adequately anchored in the connection. In addition, the lap splice length is insufficient. The average concrete strength of the specimens was 6600 psi (45.5 MPa). The yield strength of the rebar was 62 ksi (427 MPa) and 68 ksi (469 MPa) for the transverse and longitudinal reinforcement, respectively.



NOTE:
1" = 25.4 mm

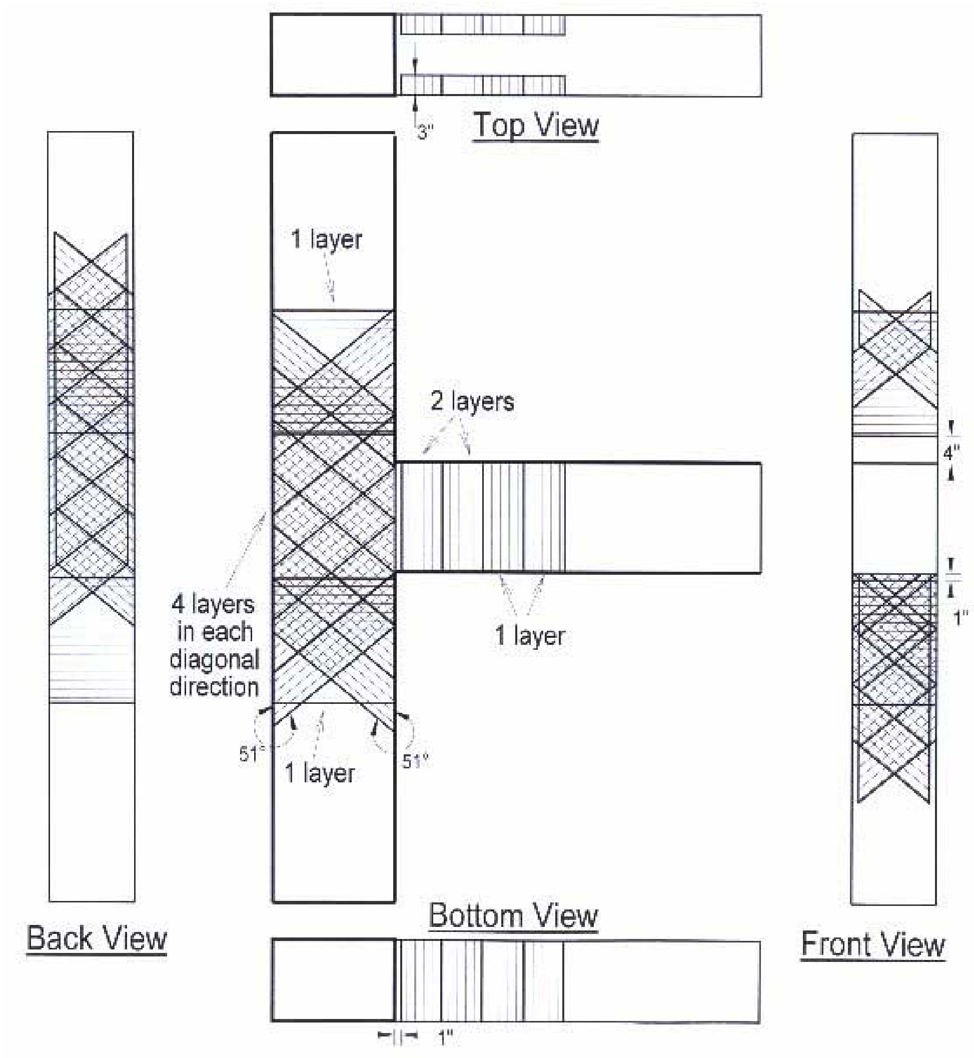
Figure 1. Specimen dimensions and reinforcement details



NOTE: 1" = 25.4 mm

Figure 2. Test setup

A schematic of the test setup is shown in Figure 2. The column was mounted horizontally, and an axial load equal to $0.1A_gf_c'$ was applied using a small hydraulic cylinder. The compression load was transferred to the specimen through four threaded rods. The lateral load was applied cyclically, in a quasi-static fashion, at the end of the beam. The push and pull force was transferred from the hydraulic actuator to the beam through a collar that fit around the beam. The actuator was pinned at the end to allow free rotation as the beam deflected.



NOTE: 1" = 25.4 mm

Figure 3. Composite retrofit layout

COMPOSITE RETROFIT DESIGN

The goal of the FRP retrofit was to improve the shear capacity of the joint, as well as increase the ductility. From the test results of the as-is specimen, shear in the joint was identified as the most critical for retrofit. Confinement in the beam and column were also deficient. The composite retrofit was designed to address these issues. The following properties of the FRP composite were used to design the layup: (1) the material was 48,000 fibers per tow unidirectional carbon fiber; (2) the number of tows per inch (25.4 mm) of material was 6.5; (3) the fiber sheets were 6 in. (152.4 mm) and 18 in. (457.2 mm) wide; (4) the ultimate FRP composite tensile strength was 91.1 ksi (628 MPa); (5) the ultimate strain was 0.01; (6) the modulus of elasticity of the composite was 9388 ksi (64,730 MPa). Figure 3 shows the FRP composite layout.

Shear strengthening of the joint

The design of the FRP composite for the joint region was determined based on the following procedure [Gergely and Pantelides 1998]: (a) the horizontal and vertical shear forces and consequently shear stresses in the center of the joint were calculated at the peak load; (b) the principal stresses in the joint were determined; (c) the shear angle was calculated to be 39° and therefore the optimum orientation of the fibers was found to be 51°; (d) the width and thickness of the FRP composite fabric was then designed to increase the diagonal tension capacity of the joint by 25 percent. From this procedure it was determined that four layers of carbon composite should be placed in both the ± 51° directions. The composite strips were wrapped around the joint on all four sides except where the beam framed into the column and where the floor slab would be if the joint was part of an actual building. A 1-inch (25.4 mm) gap was also left at the interface between the beam and column so they could move independently. These areas without carbon can be seen in Figure 3.

Flexural plastic hinge confinement of the column

The benefits of the composite jacket in the plastic hinge region include providing confinement of the core, and prevention of the concrete cover from spalling off which provides the longitudinal reinforcement with lateral stability. The composite layout was designed as a square jacket with twice the FRP composite thickness required for an equivalent circular jacket, as recommended from tests carried out by Seible et al. (1997). The thickness of the FRP composite layers was calculated using the expression derived in [Seible et al. 1997], in which the thickness was found to be proportional to the column dimensions, and inversely proportional to the product of the ultimate stress and ultimate strain of the composite. One layer of FRP composite was applied to the column above and below the joint region over the diagonal strips, as shown in Figure 3. This serves not only as confinement for the column, but also as a clamp-down device for the diagonals.

Flexural plastic hinge confinement of the beam

The shear capacity of the beam was found to be adequate outside the joint region. The design of the FRP composite for confinement of the plastic hinge followed the procedure described above for columns. Two layers of FRP composite were provided on the beam near the joint. One layer was also applied beside these two layers, as seen in Figure 3, to allow for gradual dispersion of stress beyond the plastic hinge zone. No flexural strengthening of the beam was attempted with the FRP composite.

EXPERIMENTAL RESULTS

The lateral loading was applied quasi-statically. The first portion of the test was load-controlled wherein the load was increased by 5 kip (22.2 kN) increments. At every load step three cycles were performed, each cycle containing a push and pull segment. After the first yielding in the rebar, the testing was changed to displacement-control. Three cycles were performed at each displacement step, and the displacement was increased as a fraction of the initial yield displacement. The test continued until the load dropped to 50% of the peak load.

Test of as-is joint

The load versus drift curve is shown in Figure 4. The first yielding occurred in a longitudinal column bar at a lateral load of 45 kips (200 N) and displacement of 0.6 inches (15.2 mm). The joint ultimately failed at a displacement of 1.8 in. (45.7 mm), which by a bilinear approximation corresponds to a system displacement ductility of 2.5. There was extensive shear cracking in the joint region and some spalling of concrete on the back of the column at the joint. There was a large crack which originated from a diagonal crack in the joint and extended up the column along a longitudinal bar. The cracks varied from 0.02 in. to 0.2 in. (0.5 mm to 5 mm) in width, the largest cracks residing in the joint.

Test of FRP composite retrofitted joint

The load versus drift curve for the retrofitted specimen is shown in Figure 5. The elongated hysteresis loops show a more ductile behavior than the specimen without FRP composite. The first yielding occurred in a longitudinal beam bar at a lateral load of 40 kips (178 N) and displacement of 0.5 inches (12.7 mm). The joint ultimately failed at a displacement of 3.64 in. (92.5 mm), which by a bilinear approximation corresponds to a system ductility of 4.2. The specimen had 2.1 in. (54 mm) of permanent deformation at the end of the test. There was extensive cracking at the interface between the beam and column. By the end of the test, the opening at the interface was 0.75 in. (19 mm) wide. The FRP composite delaminated from the column above and below the joint region, as well as in the lap splice region. The maximum strain reading in the carbon around the column was 0.15%. No splitting cracks were observed along the column, as occurred in the as-is specimen. In the as-is joint, the cover of the longitudinal column bars separated, but in the FRP composite retrofitted joint the cover remained in place. The FRP composite also provided containment and partial confinement of the joint's core concrete.

A summary and comparison of the strength and ductility results are presented in Table 1. The FRP composite retrofit increased the strength of the joint by 20% and the ultimate drift by a factor of two. This is shown graphically in Figure 6. Also note that the elastic stiffness of the joint did not appreciably increase from the as-is to the FRP retrofitted joint. However, the retrofitted specimen had a system displacement ductility 68% greater than the as-is specimen.

Table 1. Summary of test results

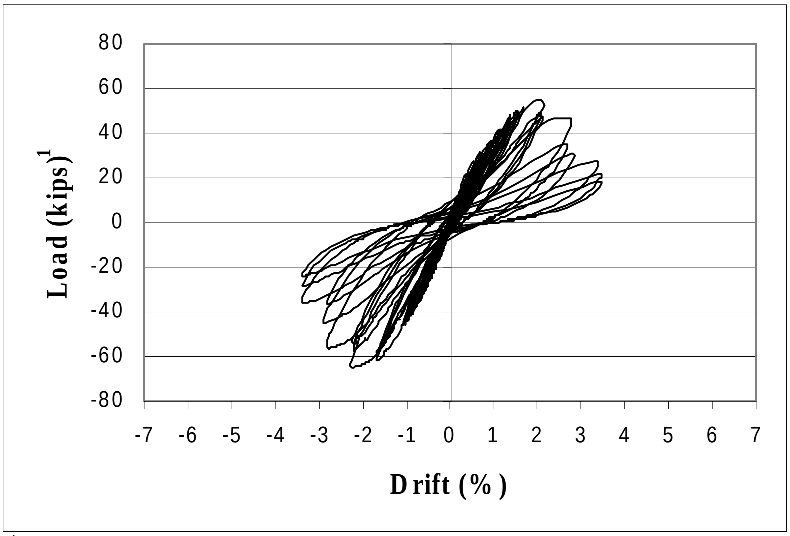
	As-is	FRP retrofit	RATIO
Peak Load	49.1 kips (218 kN)	58.7 kips (261 kN)	1.20
Drift @ 85% peak load	1.9 %	2.8 %	1.47
Absolute Maximum Drift	2.3 %	4.6 %	2.00
Ductility	2.5	4.2	1.68

By ACI 352R-91 [1991] standards, the specimen joints are considered corner joints. The code [ACI 352R-91 1991] imposes a limit on the joint shear of the as-is Type 2 joint equal to 12 $\sqrt{f_c'}\, b_j\, h$ (psi) [0.996 $\sqrt{f_c'}\, b_j\, h$ (MPa)]. However, this expression does not incorporate the effect of the axial load in the column. For beam-column joints with an axial column load but without transverse reinforcement in the joint region, the following equation for the resisting horizontal joint shear force is suggested [Park 1997].

$$V = k\sqrt{f_c'}\sqrt{1+\frac{N}{A_g\, k\sqrt{f_c'}}}\; b_j\, h \qquad (1)$$

where b_j = effective width of the joint (usually the column width), h = depth of the column, k = 0.4 for exterior joints with beam longitudinal bars that terminate as hooks into the joint, f_c' = concrete strength in MPa, N = axial load in the column, and A_g = gross cross sectional area of the column.

Using this expression, the predicted joint shear capacity of the joint is 8.1 $\sqrt{f_c'}\, b_j\, h$ (psi) [0.672 $\sqrt{f_c'}\, b_j\, h$ (MPa)]. The capacity of the as-is joint actually achieved in the test was found to be 8 $\sqrt{f_c'}\, b_j\, h$ (psi) [0.664 $\sqrt{f_c'}\, b_j\, h$ (MPa)]. This result validates Park's [1997] expression. This non-conforming joint can only achieve 66% of the ACI 352R [1991] Type 2 joint shear strength. In addition, the ACI 352R [1991] does not take the axial column load into account.



1kip = 4.45 kN

Figure 4. Load-drift hysteretic response for the as-is joint

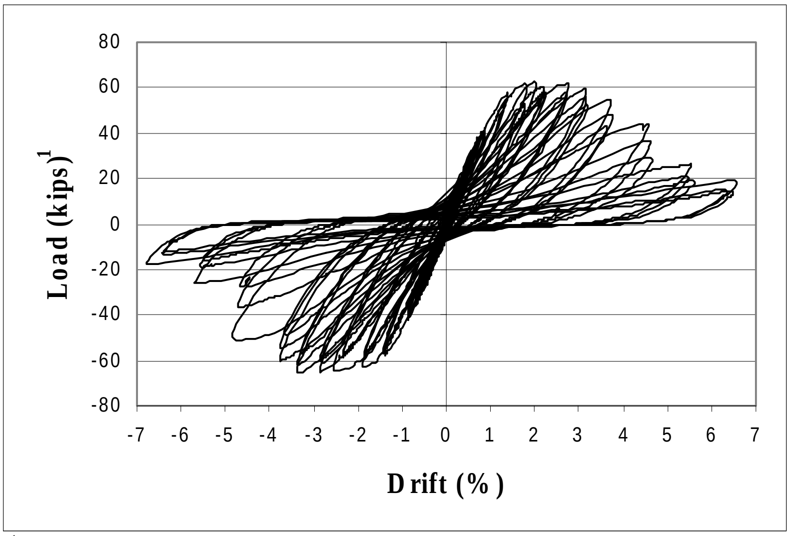


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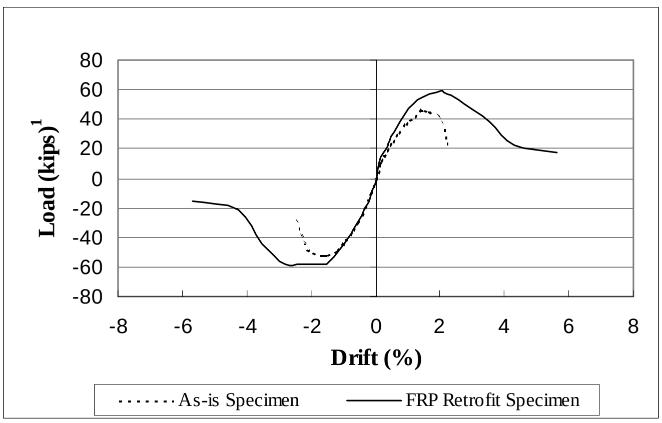
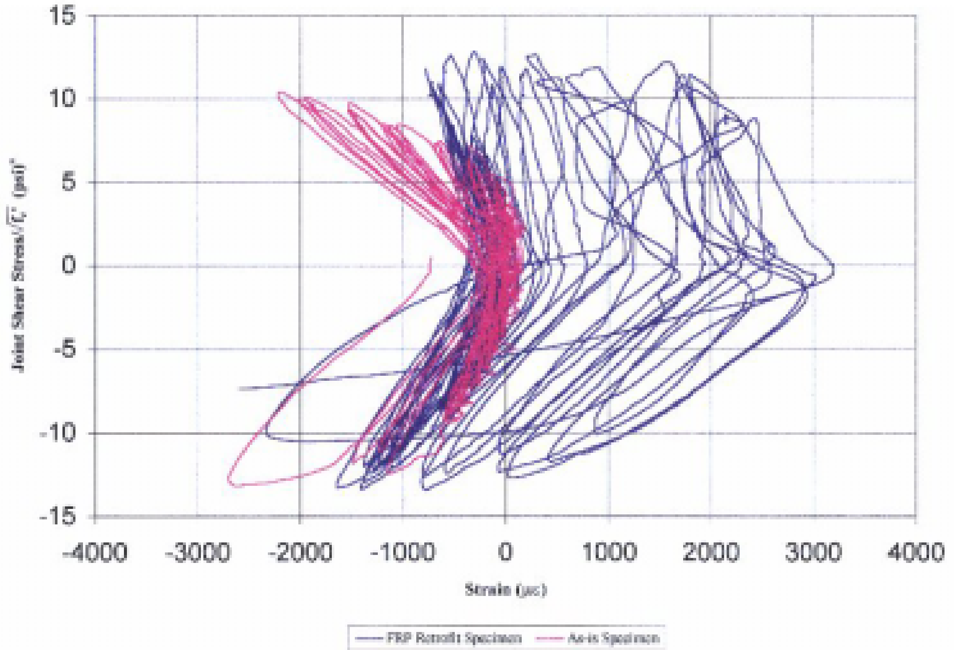
Figure 5. Load-drift hysteretic response for the retrofitted joint



1kip = 4.45 kN

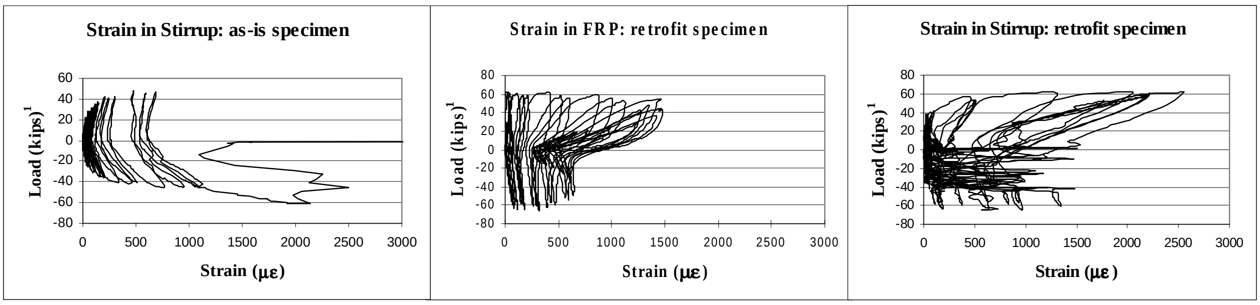
Figure 6. Backbone curves

The joint shear behavior of both specimens can be seen in Figure 7. This diagram shows that the retrofitted specimen absorbed more energy, which is proportional to the area under the loops, than the as-is specimen. The FRP composite also increased the joint shear strength by approximately 45%.



[a]1 psi = 0.0069 MPa

Figure 7. Joint shear behavior



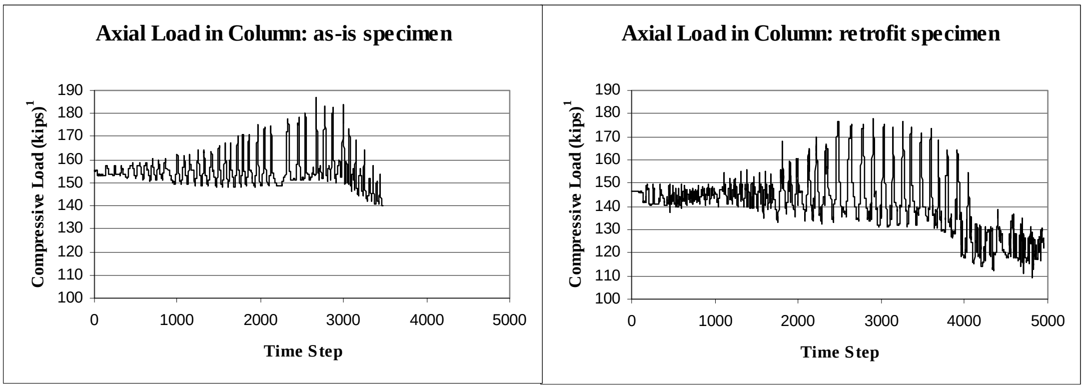
[1]1kip = 4.45 kN

(a) **(b)** **(c)**

Figure 8. Strain in hoop direction

The highest level of shear strain occurred in the hoop direction of the front face of the column just above the joint. The strain in the interior stirrup at this location is plotted for both the as-is and retrofitted specimen, as well as the strain in the carbon at the closest instrumented point. Figure 8 shows the load versus shear strain relationships. Comparing Fig. 8(a) and Fig. 8(c) , it is clear that the retrofitted specimen achieves higher strain and load levels. In addition, it is obvious from Fig. 8(b) and Fig. 8(c) that the FRP composite is sharing the load with the transverse reinforcement.

The response of the column to the cyclic loading is represented by the plot of axial column load, originally set at $0.1A_gf_c'$ shown in Figure 9. At the peak load, the as-is specimen had a 2.6% drop in axial load and a 3.9% drop at 85% of the peak load. The retrofitted specimen experienced a 8.8% loss in axial load at peak and a 9.5% loss at 85% of the peak load. At the conclusion of the test, the as-is column was carrying 90% of the initial compressive load, and the FRP specimen was carrying approximately 80%.



[1]1kip = 4.45 kN

(a) **(b)**

Figure 9. Axial load in the column

CONCLUSIONS

The above described tests of an as-is and retrofitted joint specimen show that the FRP composite can substantially improve the seismic performance of exterior R/C building joints without adequate reinforcement. The as-is joint achieved 66% of the shear strength allowed by the ACI 352R Code [1991] for Type 2 joints. The FRP composite wrap can increase the strength, ductility, and drift performance of the joint. The FRP composite alleviates splitting cracks around the outer column bars such that the concrete cover does not spall off. The FRP composite also provided containment and partial confinement of the joint's core concrete. The as-is specimen maintained 90% of the axial load in the column at a drift level of 2.3%, whereas the FRP retrofitted specimen maintained 80% of the axial load at a drift level of 4.6%. The joint shear strength of the FRP retrofitted joint was 45% higher than that of the as-is joint.

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REFERENCES

ACI Committee 352 (1991). "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures." American Concrete Institute, Report 352R-91, Farmington Hills, MI.

ACI SP-123 (1991). "Design of Beam-Column Joints for Seismic Resistance." American Concrete Institute, Report SP-123, Farmington Hills, MI.

Gergely, I., and Pantelides, C. P. (1998). "Design aid for shear strengthening of reinforced concrete T-joints using carbon fiber reinforced plastic composites." Res. Rep. UUCE 988-02, Univ. of Utah, Salt Lake City, Utah, NTIS Accession No. PB98-158504, Dec. 1998.

Park, P. (1997). "A Static Force-based Procedure for the Seismic Assessment of Existing Reinforced Concrete Moment Resisting Frames." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 30(3), 213-226.

Seible, F., Priestley, M. J. N., Hegemier, G., and Innamorato, D. (1997). "Seismic retrofitting of RC columns with continuous carbon fiber jackets." *Journal of Composites for Construction*, ASCE, 1(2), 52-62.