

Seismic Retrofit of Reinforced Concrete Beam-Column T-Joints in Bridge Piers with FRP Composite Jackets

by C.P. Pantelides and J. Gergely

Synopsis: The research described encompasses laboratory as well as in-situ testing of reinforced concrete beam-column joints and multicolumn bridge piers rehabilitated with FRP composite jackets. Fourteen RC beam-column joint tests were performed and a design equation was developed which determines the thickness of the FRP composite jacket and the orientation of the fibers for maximum effectiveness in enhancing shear capacity and ductility. Several in-situ tests were conducted at the South Temple Bridge in Salt Lake City, which included a three-column bridge pier without an FRP composite seismic retrofit, a pier retrofitted with FRP composite jackets, and a pier retrofitted with FRP composite jackets and a reinforced concrete grade beam. The design of the seismic retrofit was based on rational criteria, which included the design of the foundation and column retrofit, and the design equation for retrofitting reinforced concrete beam-column joints, developed in the laboratory tests. The performance target for the seismic retrofit was a displacement ductility twice that of the pier without the FRP composite retrofit. The FRP composite jacket was able to strengthen the cap beam-column joints of the pier effectively and the displacement ductility was increased to the designed level.

Keywords: beam-column joints; bridges; design; experiments; fiber-reinforced polymer composites; piers

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RESEARCH SIGNIFICANCE

Recent earthquakes have shown that older RC buildings and bridges are susceptible to beam-column joint failure. The beam-column joint dimensions and reinforcement details of older RC structures were determined from the sizes of adjacent framing members and the compression development length requirements for longitudinal reinforcement. Moreover, transverse reinforcement in beams and columns typically was discontinued at the face of the joint, leaving the joint without any transverse reinforcement. It is often impractical to retrofit older, lightly reinforced building and bridge beam-column joints with the volume and details of transverse steel reinforcement recommended for new structures. A more practical rehabilitation method is considered using fiber reinforced polymer (FRP) composite jackets. The paper presents a seismic rehabilitation strategy for RC building and bridge beam-column joints with deficient details using externally applied fiber reinforced polymer jackets. The method is shown to be successful in field tests of actual RC bridge piers.

INTRODUCTION

The state-of-the-art regarding repair and strengthening techniques for reinforced concrete beam-column joints has recently been reviewed by Engindeniz et al. (2005). This paper is focused on seismic rehabilitation of generic reinforced concrete (RC) joints with examples pertaining to actual bridges. Of particular interest are experimental and analytical studies regarding the seismic behavior of T-joints in multi-column bridge frames constructed in the 1950's and 1960's. The dimensions and reinforcement details of RC bridges constructed during this time make them vulnerable to failure in strong ground shaking during large earthquakes. T-joints in multi-column bridge frames, involve larger column and cap beam cross-sections, larger diameter steel reinforcement, and different geometry and anchorage conditions compared to joints in RC buildings. Moreover, in RC bridges yielding of column reinforcement is preferred rather than yielding of beam reinforcement, as is the usual case in RC buildings. The application of carbon fiber reinforce polymer (CFRP) composite jackets for the three columns and cap beam T-joints of an existing RC bridge pier, which was performed in Salt Lake City in 1996, was investigated by Gergely et al. (1998). The evaluation of the pier in the as-built condition, the rehabilitation objectives, and the CFRP wrap design were achieved by performing a pushover analysis of a structural model of the bridge pier. It was found that the shear capacity of the T-joints and the displacement ductility of the bridge pier were improved significantly. In addition, a comparison was made between analytical results and full-scale experiments of CFRP composite retrofitted T-joints carried out by Halling et al. (2001). Lowes and Moehle (1999) studied two methods for retrofitting beam-column T-joints in RC bridge structures. In the first method, addition of RC bolsters to the cap beam and joint was used; the retrofitted T-joint displayed moderate ductility capacity under simulated earthquake and gravity loading. In the second method, a post-tensioned concrete retrofit connection involving addition of post-tensioned concrete bolsters to the cap beam and joint was used; the retrofitted T-joint displayed large ductility under simulated earthquake and gravity loads. The post-tensioning improved both joint shear strength and column longitudinal reinforcement anchorage strength.

Fourteen 1/3-scale tests of RC beam-column T-joints, eleven of which were rehabilitated with CFRP composites, were carried out by Gergely et al. (2000). The variables considered were the composite system, the fiber orientation, and the surface preparation. The tests demonstrated the viability of carbon FRP composites for their use in improving the shear capacity of T-joints as evidenced by the experimental results. Based on these experimental results, a design aid was developed for T-joints with inadequate shear reinforcement. In-situ lateral load tests of three-column bridge piers were conducted on Interstate 15 to determine the strength and ductility of an existing concrete bridge and the improvements achieved using a CFRP composite retrofit by Pantelides et al. (1999). The retrofit was developed based on rational guidelines for the columns, cap beam, and T-joints to double the displacement ductility of the as-built pier. The peak lateral load capacity was increased and the displacement

ductility was improved. Two new concepts of strengthening multi-column bridge piers with CFRP composites were identified by Pantelides et al. (2001): the first involved shear strengthening of a T-joint through a CFRP composite "ankle-wrap"; the second concept was a CFRP composite "U-strap" to improve the anchorage of column longitudinal steel reinforcement extending into the joint. The performance of a multi-column bridge pier that was tested in the as-built condition first, repaired with epoxy injection and CFRP composite jackets and re-tested was also described. Recommendations for improvement of the original CFRP composite seismic retrofit design were offered based on the lessons learned from the in-situ tests (Pantelides and Gergely 2002).

BEAM-COLUMN JOINT COMPONENT LABORATORY TESTS

In laboratory tests of beam-column joints without transverse reinforcement, (Gergely et al. 2000), the CFRP composite retrofit was applied with the layout shown in Fig. 1. A carbon fiber fabric was used for all the specimens; two types of resin were used: an elevated temperature cure two-component epoxy which required a cure temperature of 154 °C (309 °F), and an ambient temperature cure two-component epoxy. The elevated temperature cure was carried out for 90 minutes with a cooling ramp of 30 min. The ambient cure was carried out for a period of at least 7 days. All resin curing schedules were as recommended by the manufacturer, and all tests were carried out long after complete cure had taken place. The elevated temperature CFRP composite had a tensile strength of 4,300 MPa (624 ksi) and an elastic modulus of 230 GPa (33,360 ksi). The ambient cure CFRP composite had an ultimate tensile strength of 1,160 MPa (168 ksi) and an elastic modulus of 85 GPa (12,330 ksi). CFRP composite sheets were applied at ±45 degrees at the top, side and bottom faces of the T-joint as shown; in addition CFRP composite layers were applied to confine the column in the vicinity of the joint (L1), and additional layers were applied around the cap beam (L3) for shear strengthening of the cap beam and anchorage enhancement of the CFRP composite layers applied at ±45 degrees. The specimens were tested in the inverted position under quasi-static loads applied perpendicular to the column. The damage observed for the as-built joints was extensive resulting in joint damage and shear failure, while the damage observed for rehabilitated joints was gradual delamination of the CFRP sheets from the beam faces resulting in a contained damage of the concrete as shown in Fig. 1. Typical hysteresis loops are shown in Figs. 2, 3 for the as-built and rehabilitated joints. A 60% increase in strength and a 100% increase in displacement ductility were observed for certain specimens.

RC MULTICOLUMN BRIDGE PIER FIELD TESTS

Field tests of RC bridge multi-column bridge piers were carried out at the South Temple Bridge on Interstate 15 in Salt Lake City, during 1998 and 2000. The South Temple Bridge was built in 1962 without the seismic details required by current seismic codes. The dimensions of the as-built pier and the loading condition for the 1998 tests are shown in Fig. 4. The steel reinforcement details, which were identical for both the 1998 and 2000 tests, are shown in Fig. 5. There were no vertical stirrups or horizontal hoops present in the T-joints; in addition, there was no shear steel reinforcement in the pile caps and no top steel reinforcing bars. AASHTO (2002) requires that column transverse reinforcement for confinement be provided at a maximum of 100 mm (4 in.) for the top and bottom 1.22 m (48 in.) column height; however, the 13 mm (#4) single hoops provided were spaced at 305 mm (12 in.) and did not meet current standards; the cross-sectional area of the transverse reinforcement was only 43% of the area required by current standards. Lap splices are permitted only within the center half of the column height and the splice length should not be less than 60 steel bar diameters; the splice was in the plastic hinge region at the column base and the splice length was only 24 bar diameters. The requirement for column hoops of a closed tie with 135-degree hooks having a 76 mm (3 in.) extension at each end were not met by the reinforcement. The development length of column reinforcement into the cap beam required by AASHTO was 0.89 m (35 in.); this was met since the reinforcement extended 0.91 m (36 in.) into the cap beam but the details did not meet the current standards for column transverse reinforcement extending a distance 0.38 m (15 in.) from the face of the column connection into the adjoining members; no transverse reinforcement was provided in the T-joints. ACI Committee 318 report (2005) states that cap beam positive moment capacity at the T-joint face should not be less than one-half the negative moment strength provided at the joint face. The ratio of positive to negative moment strength was 0.35, which does not meet current design recommendations.

As-built and retrofitted bridge piers – 1998 tests

The piles were 305 mm (12 in.) concrete filled steel pipes and had an average length of 19.81 m (65 ft); they were embedded into the 0.914 m (36 in.) thick pile cap a distance of 0.305 m (12 in.) as shown in Fig. 6. The

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connection of the piles to the pile cap consisted of four 19 mm (#6) bars which extended only 0.305 m (12 in.) into the pile cap, as shown in Section A-A of Fig. 6. The capacity of these bars would be exceeded for the lateral loads anticipated in the test. In addition, the existing short bar anchorage would cause pullout failure of the piles. A 38-mm (1 1/2 in.) hole was cored through the pile cap and into the pile for a distance of 2.438 m (8 ft) as shown in Fig. 6. The pile was anchored into the pile cap using a 32-mm (1 1/4 in.) Dywidag bar epoxied into the hole. This detail was implemented for the four corner piles of the pile caps. The piles did not have adequate shear capacity to carry the lateral loads anticipated in the tests; for this reason, two 36 mm (1 7/16 in.) Dywidag bars were connected by anchoring their ends to a wide flange beam on one side, and by casting them in the load frame footings on the other, as shown in Fig. 4. To minimize lateral movement of the pile caps and complete the load path, the existing RC strut shown in Fig. 5(a) was widened with two sections of unreinforced concrete to carry the compression force; in addition, a 0.510-m (20 in.) long concrete strut reinforced with four 25-mm (1 in.) longitudinal bars, was used to join the pile cap to the load frame footing, as shown in Fig. 4. In the tests, a horizontal quasi-static cyclic load was applied at the cap beam level, as shown in Fig. 7. Significant steel corrosion was measured in the cap beam; the stirrups had an average loss of area in the vertical legs between 12% and 16%; surface preparation involved removal of the cap beam cover and application of shotcrete to establish a concrete surface that ensured good bond of the CFRP composite. To pull the pier, the load was transferred to the far end of the cap beam by twenty 13-mm (1/2 in.) diameter prestressed tendons, that were stressed to only 6 MPa (870 psi) to take out the sag in the tendons and the slippage in the anchorage system. The load-displacement hysteresis curve for the as-built pier is given in Fig. 8; the maximum load reached was 1,566 kN (352.1 kips) and the maximum displacement was 137 mm (5.39 in.), with a displacement ductility of 2.8. Diagonal tension cracks developed in the joints and the longitudinal bars in the cap beam buckled; in addition, flexural cracks developed at the top of the columns in the plastic hinge region and radial cracks developed at the top of the pile cap around the column perimeter.

A second pier was retrofitted similar to the as-built pier with the addition of CFRP composites; the CFRP composite layout for one of the columns of the retrofitted pier is shown in Fig. 9. The design of the CFRP composite for column strengthening followed established procedures (Seible et al. 1997). A 51-mm (2 in.) gap was left between the column and pile cap, and the column and cap beam, to avoid any stiffness increase from the retrofit. At the bottom of the column, there were ten layers of CFRP composite (L10) extending to a distance of 914 mm (36 in.) primarily for lap splice clamping, and the remainder for flexural plastic hinge confinement and shear strengthening. Next, two layers were placed (L2) for a distance of 457 mm (18 in.) for flexural plastic hinge confinement and shear strengthening, and finally one layer (L1) was placed for flexural plastic hinge confinement. The CFRP jacket at the top of the column did not require lap splice clamping and has fewer CFRP composite layers. Two continuous layers of CFRP composite were placed at the cap beam in the form of vertical hoops on both sides of the column, as shown in Fig. 9, for flexural hinge confinement and shear strengthening of the cap beam. The design of the CFRP composite for strengthening the cap beam-column T-joints was achieved with the following procedure. The T-joint principal tension stress σ_x and average joint shear stress τ_{xy} were obtained using a lateral load pushover analysis of the pier, with a monotonic lateral load applied at the cap beam level; the resulting principal angle, θ_p , is given as:

$$\cos 2\theta_p = \frac{\sigma_x}{2\sqrt{\left(\frac{\sigma_x}{2}\right)^2 + (\tau_{xy})^2}} \quad (1)$$

To maximize the contribution of the CFRP composite, the CFRP composite fabric fibers were placed at an angle of $\theta = (90^\circ - \theta_p)$ from the longitudinal axis of the cap beam; in the in-situ application the sheets were laid at $\pm 45^\circ$ for practical reasons. A second lateral load pushover analysis of the retrofitted bridge pier was carried out which resulted in an increased demand for the joint principal tensile stress, that was used to determine the number of CFRP composite layers in the T-joint. A diagonal tension crack in the joint was analyzed to find the number of CFRP composite layers inclined at 45° required to increase the capacity of the joint in principal tension. Figure 10 shows the cap-beam column T-joint with the composite layer perpendicular to the crack. Force F_2 acting normal to the crack is resisted by one CFRP composite layer stressed in the fiber direction. The magnitude of F_2 is:

$$F_2 = \phi_v t_j \varepsilon_j E_j \frac{d_e}{\cos \theta} \quad (2)$$

where ϕ_v = shear strength reduction factor, assumed as 1.0; t_j = thickness of one composite sheet; ε_j = average axial strain of the composite in the fiber direction at peak horizontal load, which was taken as 2 mm/m (0.002 in./in.); and d_e = effective T-joint depth equal to the T-joint height minus twice the effective bond length of the composite sheet to the concrete; from laboratory tests this was approximately 51 mm (2 in.). To find the tensile stress σ_f developed in one FRP composite layer, force F_2 is divided by the product of the T-joint width b and inclined length along the crack as:

$$\sigma_f = \frac{F_2 \cos \theta}{b d_e} \quad (3)$$

A number of layers, each of capacity σ_f , must be used to resist the increase in principal tensile stress demand from the as-built to the retrofitted pier equal to $\Delta\sigma$. The total number of layers required is then obtained as:

$$Ln = \frac{\Delta\sigma}{\sigma_f} \quad (4)$$

Two CFRP composite layers (L2) were provided on both sides of the cap beam at $\pm 45^\circ$, i.e. a total of four layers in the $+ 45^\circ$ direction and four layers in the $- 45^\circ$ direction, as shown in Fig. 9. Six layers of 152 mm (6 in.)-wide CFRP composite straps (L6) were provided at the left and right side of the column over the cap beam to form the U-strap, whose design was based on the increased tensile force demand of the retrofitted pier at the T-joints. The U-straps were clamped using one layer of CFRP composite (L1) wrapped around the column, over the straps in the transverse direction, as shown in Fig. 9. The two exterior columns and adjacent portions of the cap beam and the two exterior cap beam-column T-joints were identical to the interior column and T-joint retrofit. The properties of the ambient temperature cured CFRP composite were determined according to ASTM D-3039 (ASTM 1996) as: modulus of elasticity = 65 GPa (9,430 ksi), tensile strength = 628 MPa (91 ksi), ultimate axial strain = 10 mm/m (0.01 in./in.), layer thickness = 1.32 mm (0.052 in.), and fiber volume fraction = 35%. The lateral load-displacement hysteresis curve for the retrofitted pier with CFRP composites is given in Fig. 11; the maximum load reached was 1,960 kN (440.6 kips) and the maximum displacement was 265 mm (10.43 in.), with a displacement ductility of 6.3. Thus, the goal of doubling the displacement ductility of the pier without the CFRP composite retrofit was achieved, as can be observed by comparing Figs. 8 and 11. Large cracks at the interface of the columns with the cap beam were observed and the CFRP layers delaminated in the joint regions; in addition, the CFRP U-straps failed in tension. A more extensive treatment of the 1998 tests is given in Pantelides et al. (1999), Gergely et al. (2000), Pantelides et al. (2001), and Pantelides and Gergely (2002).

Retrofitted bridge pier – 2000 tests

The loading condition for the 2000 tests is shown in Fig. 12; it is different from that of the 1998 tests since the bridge foundation is independent of the load frame foundation as can be seen by comparing with Fig. 4. The seismic rehabilitation of the foundation is shown in Fig. 13. The piles were connected to the pile cap with four 19 mm (#6) steel bars extending 0.305 m (12 in.) into the pile cap. For the expected ultimate lateral load in the test, the area and anchorage length of the existing steel reinforcement would have been insufficient to resist pullout failure of the piles. A 38-mm (1-1/2 in.) hole was cored through the pile cap into the pile for a distance of 2.44 m (8 ft), and the pile was anchored to the pile cap using a 32 mm (1-1/4 in.) Dywidag bar epoxied into the hole, as shown in Fig. 13(c); this detail was implemented for the four corner piles of each pile cap. As part of the foundation rehabilitation, a RC grade beam was constructed consisting of two 0.84 m (33 in.) x 0.46 m (18 in.) RC beams on the sides of the existing 0.46 m (18 in.) square RC strut, along with a 0.305 m (12 in.) thick by 2.13 m (7 ft) wide RC overlay with two end beams at the edges of the exterior pile caps, as shown in Fig. 13. The design of the RC grade beam can be found in Pantelides et al. (2004). The RC grade beam caused the three pile caps to displace uniformly and increased the shear and flexural capacity of the foundation. The two 0.84 m x 0.46 m (33 in. x 18 in.) RC beams

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adjacent to the RC strut as shown in Fig. 13(b) provided the compression load path; the 0.305 m (12 in.) RC overlay along with the two end beams provided the tension load path. The longitudinal steel reinforcement consisted of two layers of 12 x 25 mm (1 in.) bars in the 0.305 m (12 in.) overlay, and two groups of 3 x 25 mm (1 in.) steel bars along the bottom between the pile caps. The shear reinforcement was 10 mm (#3) steel stirrups spaced at 152 mm (6 in.) over the pile caps and end beams as shown in Fig. 13; the stirrup spacing was increased to 406 mm (16 in.) between the pile caps as shown in Fig. 13(a). Special hoop steel reinforcement was used for the two end beams, shown in Fig. 13(d), whose purpose was to anchor the tension tie made up of the top steel reinforcement at the ends of the exterior pile caps.

The retrofitted bridge pier tested in 2000 was rehabilitated with CFRP composites to increase the displacement ductility of the as-built pier by a factor of two. The CFRP design for the columns followed the retrofit concepts used in the 1998 tests. Four FRP composite elements were identified for the T-joint rehabilitation as shown in Fig. 14: (1) Element 1 consisted of horizontal CFRP layers (zero layers) applied at the two sides of the cap beam along the cap beam axis in the East and Middle T-joints for shear and flexural strengthening of the T-joint; (2) Element 2 was the diagonal CFRP ankle wrap applied at top, side and bottom faces of the T-joint for resisting diagonal tension; (3) Element 3 was placed for confining the flexural hinge and increasing the cap beam shear using CFRP vertical hoops on both sides of the T-joints; (4) Element 4 consisted of CFRP U-straps from the columns to the top of the cap beam to anchor the vertical column steel bars terminating 0.31 m (12 in.) from the cap beam top, as shown in Fig. 5(b).

The design of the CFRP layers parallel to the cap beam axis (zero layers) for element 1 and the CFRP U-straps for element 4 was governed by combined tensile and flexural stresses in the FRP composite and is presented elsewhere (Duffin 2004). The unidirectional fibers in the case of the zero FRP composite layers are applied in the direction of the cap beam axis; the unidirectional fibers in the case of the FRP composite U-straps are in the direction of the column axis. The design of the ankle wrap followed the procedure of Eqs. (1-4). For element (3), the shear capacity of the cap beam was found to be adequate and only confinement of the potential flexural hinge regions in the cap beam near the T-joints was considered. The rehabilitation of the columns with FRP composite jackets enables them to resist higher axial loads, moments and shears; to balance the additional column strength it is necessary to increase the strength of the cap beam regions on both sides of the T-joint as shown in Fig. 14 (c). The CFRP composite vertical hoops were designed as a jacket for a rectangular section, using twice the FRP thickness required for the equivalent circular jacket of diameter, as proposed by Seible et al. (1995). The CFRP composite properties, composed of carbon fabric fibers with epoxy resin were obtained from tensile coupons manufactured on-site (ASTM D3039 1996); each FRP composite layer was 1 mm thick. The tensile strength of the CFRP composite, which had a fiber volume of 25%, was 720 MPa (104 ksi), the modulus of elasticity was $E_f = 70$ GPa (10,150 ksi), and the ultimate tensile strain was 9 mm/m (0.009 in./in.). A wet lay-up process was used to apply the CFRP composite using a saturating machine, which ensured consistent fiber impregnation with resin.

The actual CFRP composite application is shown in Fig. 15. Each column had an identical pattern of CFRP jackets designed for: (1) flexural hinge confinement, (2) lap splice clamping, and (3) shear strengthening. A 50 mm (2 in.) gap was left above the pile cap, after which 14 layers of FRP composite extended 0.91 m (36 in.), followed by three layers for an additional 0.46 m (18 in.) and finally two layers for 0.46 m (18 in.). The RC grade beam was built after the FRP composite was applied; a 13-mm (1/2 in.) thick plastic spacer was used to separate the FRP jacket from the grade beam concrete, as shown in Fig. 13(a), to prevent damage to the CFRP jacket. The column tops were rehabilitated with FRP jackets for flexural hinge confinement and shear strengthening; a 50 mm (2 in.) gap was left below the cap beam, six layers of FRP composite extended 0.91 m (36 in.), three layers 0.46 m (18 in.), and two layers 0.46 m (18 in.). CFRP composite jackets in the T-joints were provided using the four elements of Fig. 14. The CFRP composite for element 1 for the east T-joint was composed of two sheets: a 1.21 m (48 in.) high and 4.57 m (15 ft) long sheet, and a 1.21 m (48 in.) high and 3.35 m (11 ft) long sheet on both faces of the cap beam in the cap beam axis direction; for the middle T-joint there was a single 1.21 m (48 in.) high and 3.35 m (11 ft) long sheet on both faces, as shown in Fig. 15(a); there was no element 1 for the west T-joint. The CFRP ankle wrap for element 2 consisted of two layers at $\pm 52^\circ$ from the horizontal on both vertical sides of the cap beam-column joints. Each layer was 0.46 m (18 in.) wide and covered the top, side, and bottom faces of the cap beam. For element 3, the CFRP vertical hoops were placed 360° around the cap beam and consisted of four FRP layers for the first 0.46 m (18 in.) adjacent to both sides of the column, and two layers for 0.46 m (18 in.) beyond the first group of four layers, as shown in Fig. 15(a). The reinforcement for element 4 consisted of five continuous CFRP layers of two 0.36 m (14 in.) wide U-straps; the CFRP U-straps were applied at one face of the column 0.91 m (36 in.) below the cap beam

soffit reaching the other face for the same length with the fibers in the vertical direction, as shown in Fig. 15(b); these CFRP U-straps were clamped by two 0.91 m (36 in.) wide CFRP layers around the column with the fibers in the hoop direction.

The hysteresis loops of the retrofitted bridge pier tested in 2000 are shown in Fig. 16. The maximum movement of the RC grade beam was 44 mm (1.73 in.); the maximum system displacement was 521 mm (20.51 in.) in push and 115 mm (4.53 in.) in pull, achieving a displacement ductility of 5.8. The maximum lateral load in push was 2,406 kN (541 kips). The asymmetrical horizontal displacement was a result of the limited stroke of the actuator of 762 mm (30.00 in.) and the following factors: (1) horizontal movement of the load frame foundation, and (2) elongation of prestressing cables during the pull cycles. The peak horizontal displacement of the load frame foundation shown in Fig. 12 was 102 mm (4.01 in.) due to the geopier flexibility (Pantelides et al. 2002). The structure was symmetric regarding its dimensions and reinforcement, and since the observed damage did not produce any pinching of the hysteresis loops, if the pier were pushed the same amount in both directions, the outcome would have been essentially the same; this type of hysteretic response is often observed in shake table tests (Correal et al. 2002).

Degradation of the lateral load capacity of the bridge pier started once the CFRP U-straps ruptured in the T-joints of the two exterior columns at a 4% drift ratio, as shown in Fig. 17(a). Subsequently, the vertical column steel started slipping at the East and West column T-joints near a drift ratio of 5.5%, as shown in Fig. 17 (b). A flexure/shear crack formed starting at the top of the cap beam, 1.73 m (68 in.) east of the West column, as shown in Fig. 18; this is the termination point of the four 32-mm (#10) negative moment steel bars, as shown in Fig. 5(a). The crack started as a flexural crack at a 4.0% drift ratio, continued to propagate diagonally and had completely developed at a drift ratio of 6%, reaching a crack width of 1.3 mm (0.05 in.). The crack occurred because of the high negative moment and insufficient tensile steel and vertical stirrups at this location; there were no FRP composite layers parallel to the cap beam axis near this joint, which were provided for the East and Middle columns, as shown in Fig. 15(a), which did not develop any cracks. It should be stressed that the retrofit was successful, the pier dissipated a large amount of energy and reached a drift ratio of 6.8%. A more extensive treatment of the 2000 bridge pier tests is given in Pantelides et al. (2002), Pantelides et al. (2004), and Duffin (2004).

CONCLUDING REMARKS AND RECOMMENDATIONS

Seismic retrofit of beam-column joints typical of those present in RC buildings and multi-column bridge piers was investigated. Laboratory tests demonstrated that an FRP composite retrofit can increase both the strength and ductility of such joints, which typically have inferior steel reinforcement details that do not meet current seismic standards. The retrofit of existing RC multi-column bridge piers has been shown to be feasible from field tests of retrofitted bridge piers. This was possible, even though the details of the steel reinforcement did not meet current seismic standards, and corrosion of the steel reinforcement in the cap beams had occurred. It should be noted that if the details of the steel reinforcement are severely deficient, or corrosion has reached an advanced stage, seismic retrofit may not be possible and replacement may be the only option. The field tests of the RC multicolumn bridge piers demonstrated that any seismic rehabilitation scheme should include the foundation in order to be successful (Pantelides et al. 2004). Conventional strengthening techniques such as pinning the piles to the pile caps using steel dowels, and a RC grade beam overlay connecting the pile caps were essential in the success of the seismic retrofit of the bridge piers. Carbon FRP composite jackets were employed for strengthening and confinement of the columns, cap beam, and T-joints. A sequential seismic retrofit design of the bridge is necessary to achieve the desired results; brittle failures could occur by strengthening one portion of the structure and relatively weakening another. The successful seismic retrofit concepts presented in this paper could be used in the seismic rehabilitation of RC buildings and bridges built in the 1950's and 1960's for strengthening and displacement ductility enhancement.

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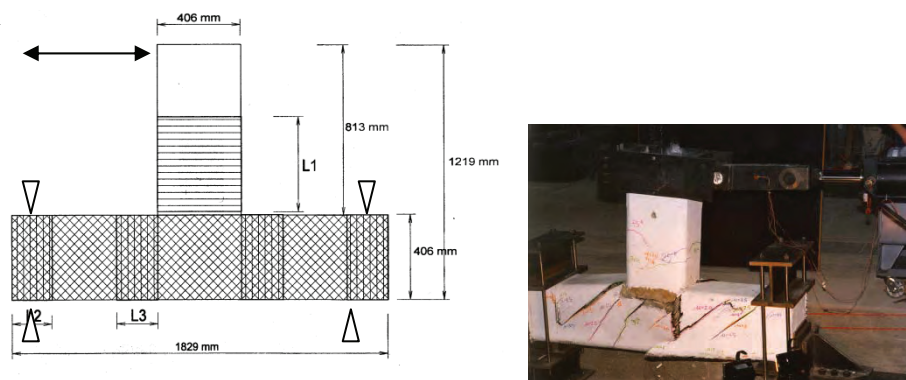


Figure 1 - CFRP composite layout for component beam-column joints (1 mm = 0.039 in.).

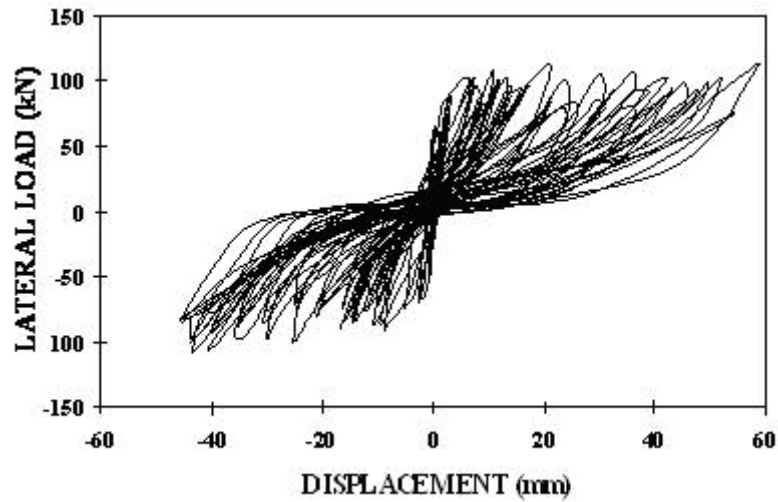


Figure 2 - Hysteresis for as-built beam-column joint.
(1 kN = 0.225 kips, 1 mm = 0.039 in.)

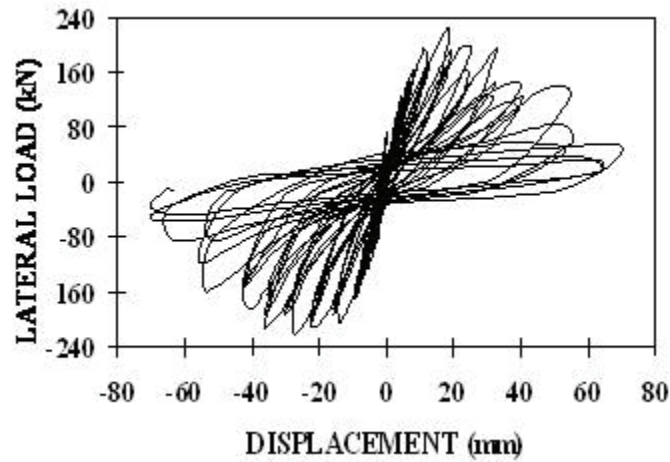


Figure 3 - Hysteresis for beam-column joint rehabilitated with CFRP composites.
(1 kN = 0.225 kips, 1 mm = 0.039 in.)

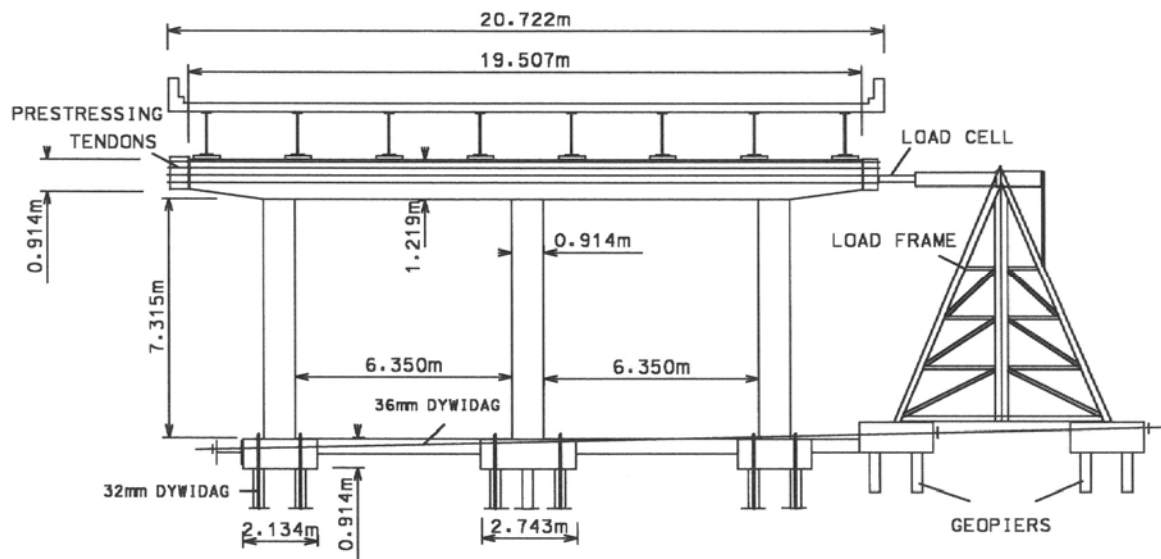


Figure 4 - Dimensions and loading condition for typical bridge pier - 1998 tests.
(1 m = 39.37 in., 1 mm = 0.039 in.)

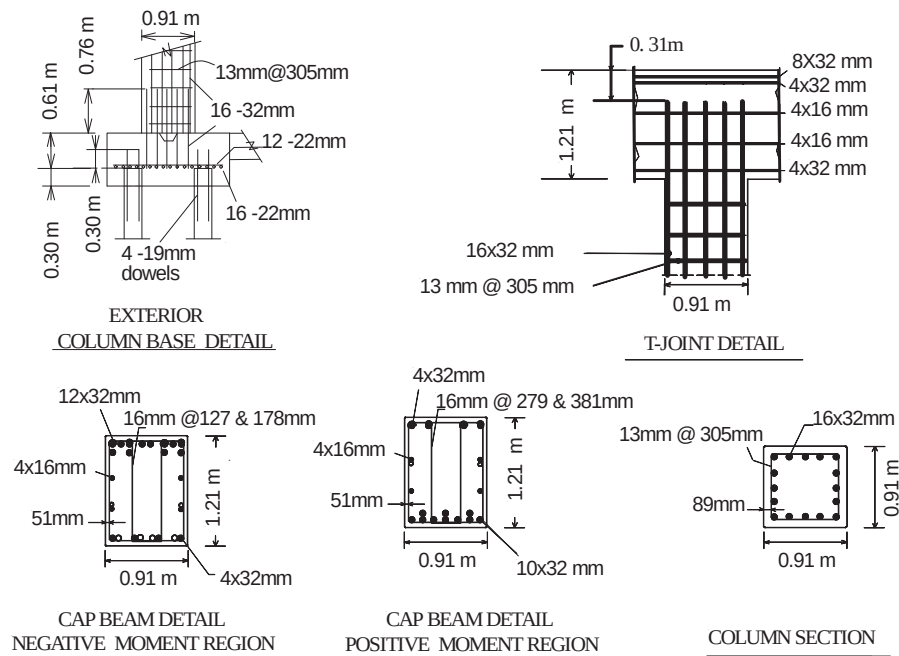
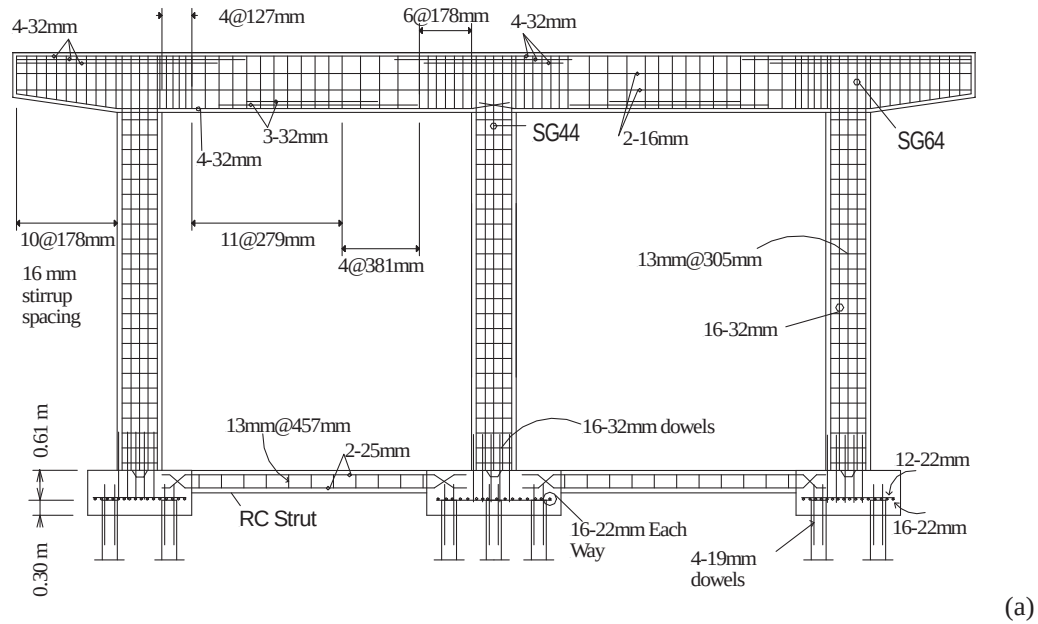


Figure 5 - Steel reinforcement of bridge pier: (a) overall details, (b) cross section details.
(1 m = 39.37 in., 1 mm = 0.039 in.)

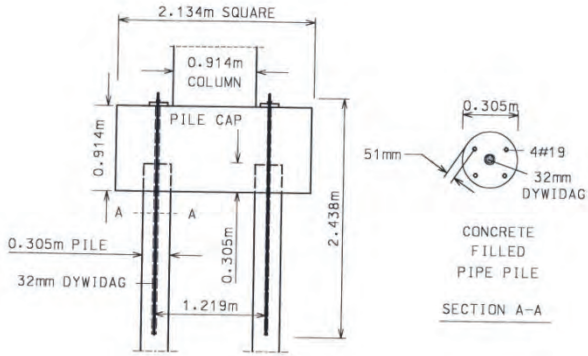


Figure 6 - Detail of pile-to-pile cap connection – 1998 tests.
(1 m = 39.37 in., 1 mm = 0.039 in.)



Figure 7 - In-situ lateral load test of bridge pier – 1998 tests.

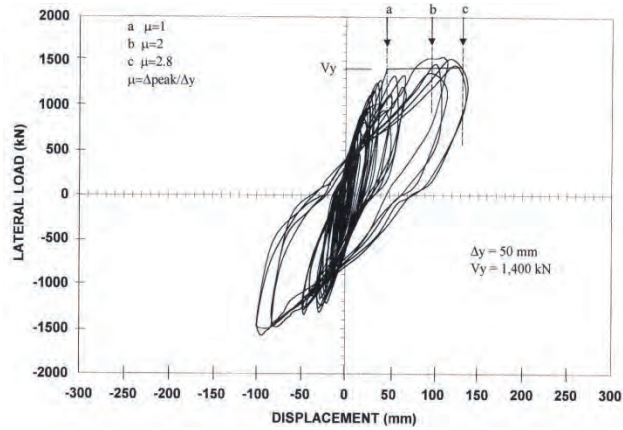


Figure 8 - Hysteresis for as-is pier – 1998 tests. (1 kN = 0.225 kips, 1 mm = 0.039 in.)

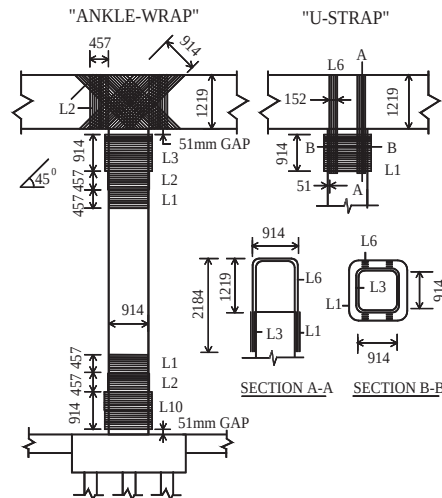


Figure 9 - Details of CFRP composite layout for retrofit of bridge pier – 1998 tests.
(1 mm = 0.039 in.)

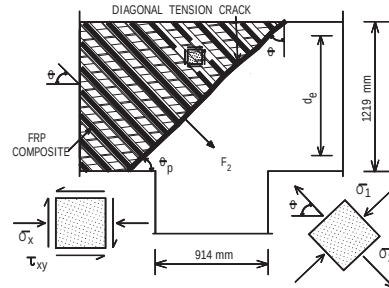


Figure 10 - Design of ankle wrap seismic retrofit of bridge T-joints using CFRP composites. (1 mm = 0.039 in.)

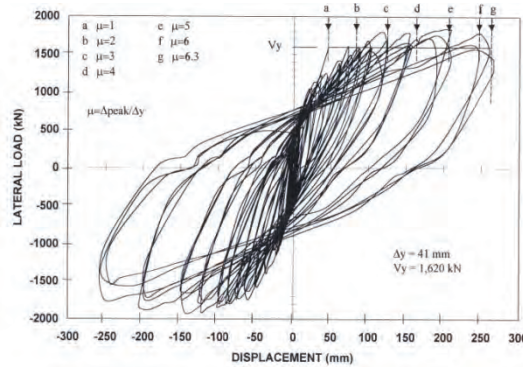


Figure 11 - Hysteresis for CFRP retrofitted pier - 1998 tests.
(1 kN = 0.225 kips, 1 mm = 0.039 in.)

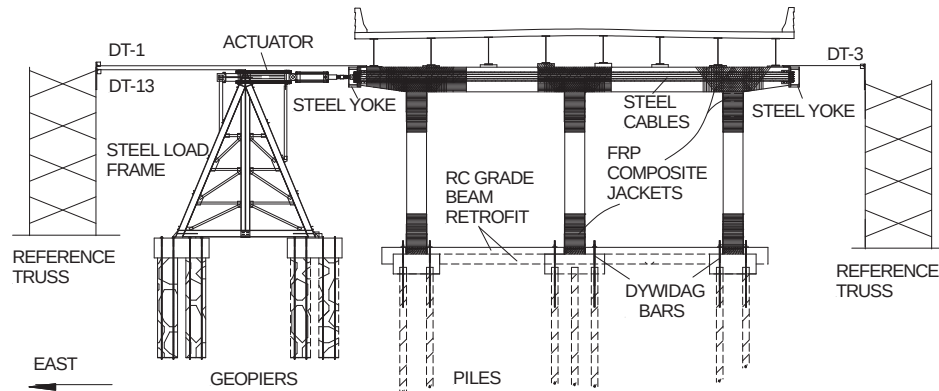


Figure 12 - Loading condition for typical bridge pier - 2000 tests.

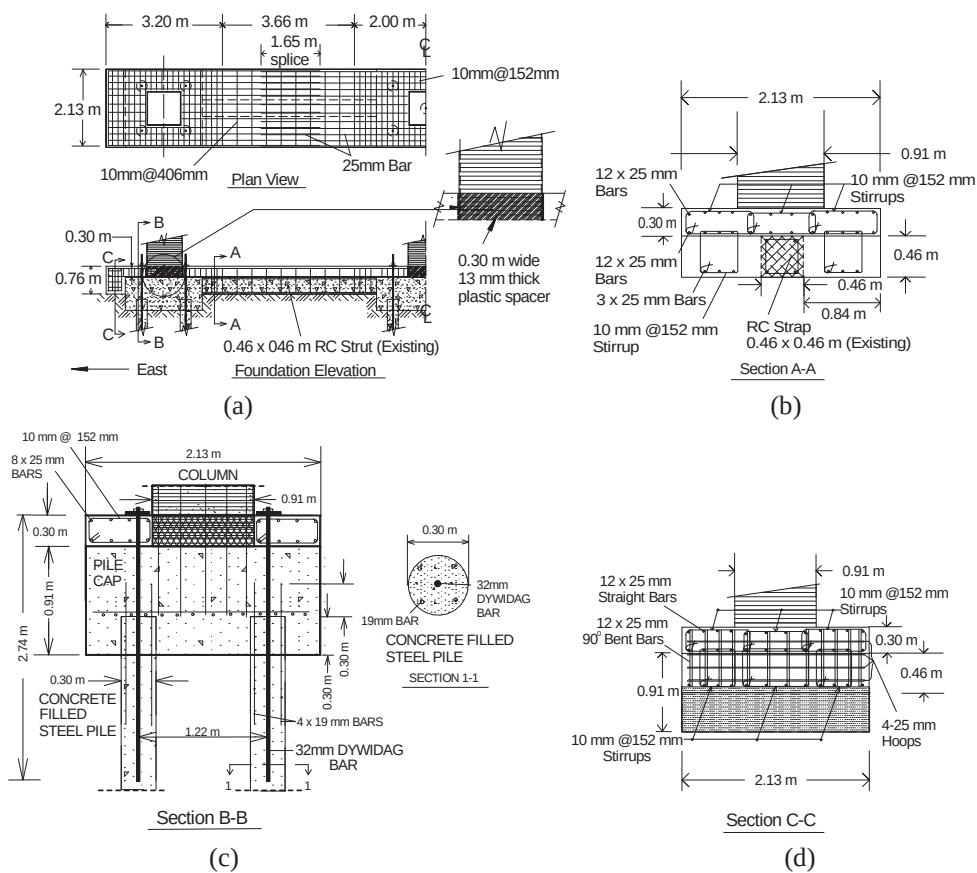


Figure 13 - Grade beam for 2000 tests: (a) plan and longitudinal section, (b) grade beam section A-A, (c) grade beam-to-pile section B-B, (d) end beam section C-C.
(1 m = 39.37 in., 1 mm = 0.039 in.)

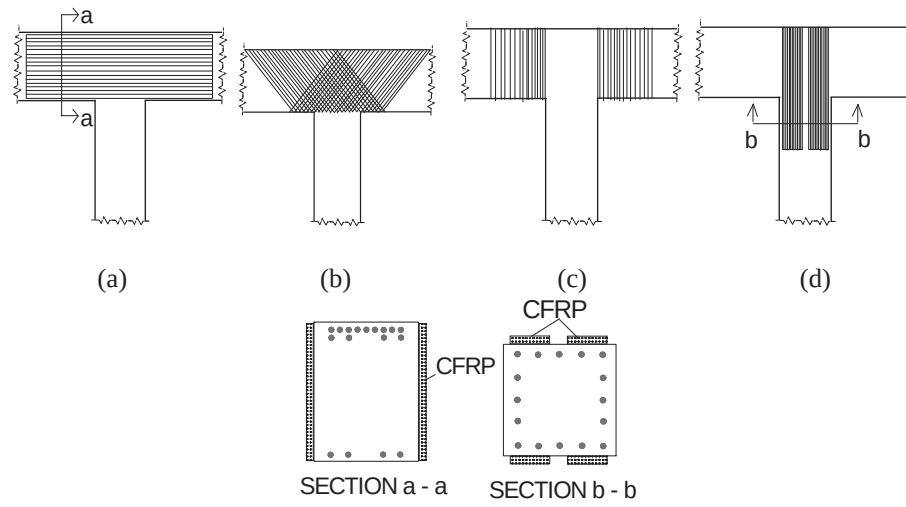
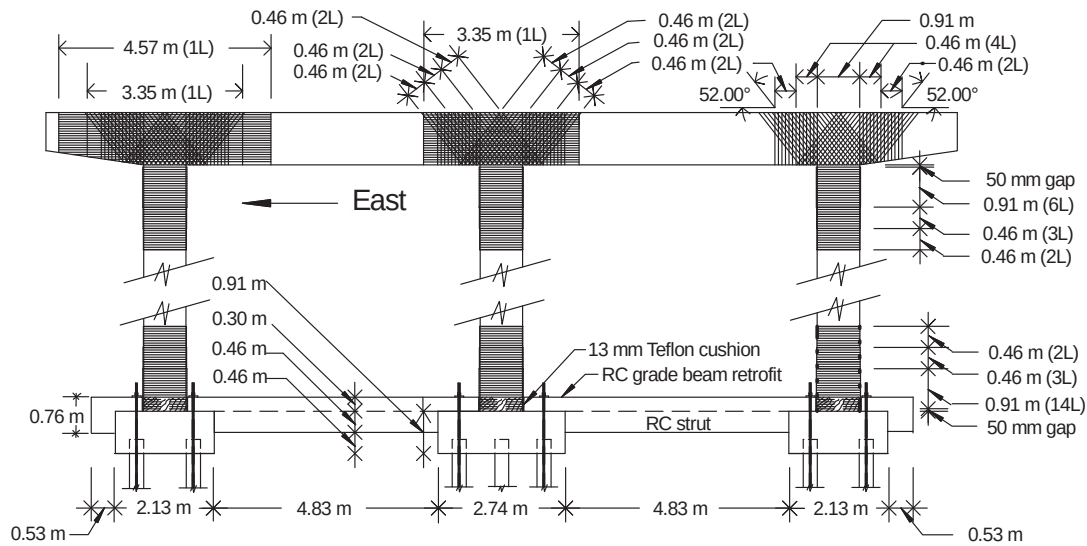
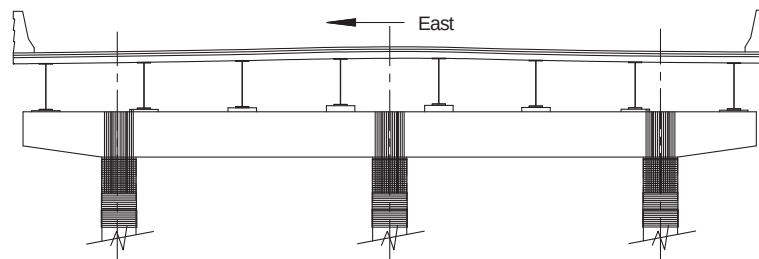


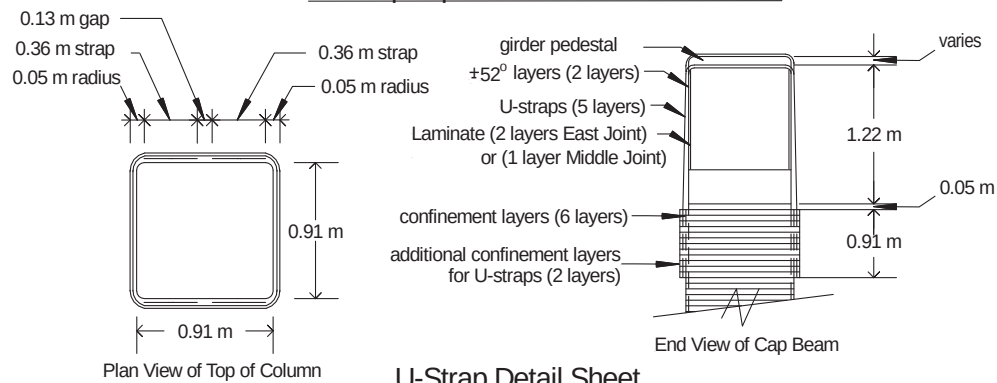
Figure 14 - Four elements of rehabilitation of T-joints with FRP composites for 2000 test: (a) zero layers, (b) diagonal ankle wrap layers, (c) hoops, and (d) U-straps.



(a)



U-Strap Top of Pier #6S - 2000 Elevation



U-Strap Detail Sheet

(b)

Figure 15 - CFRP composite design for 2000 test: (a) columns and cap beam, (b) CFRP U-straps.
(1 m = 39.37 in., 1 mm = 0.039 in.)

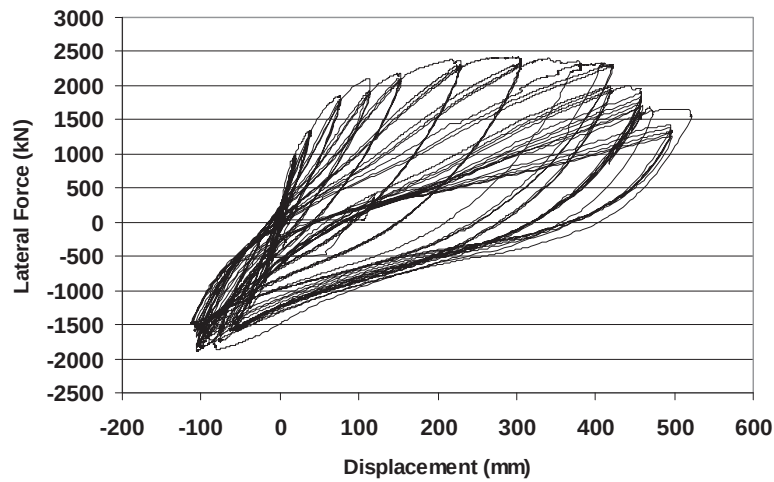


Figure 16 - Hysteresis loops for pier with RC grade beam and CFRP retrofit – 2000 tests.
(1 kN = 0.225 kips, 1 mm = 0.039 in.)

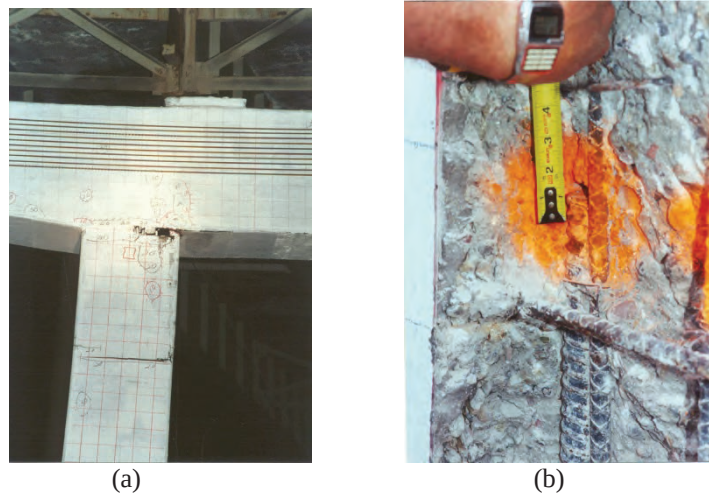


Figure 17 - Damage of retrofitted bridge pier in 2000 tests: (a) CFRP U-Strap rupture at 4% drift ratio at East column T-joint, (b) 16 mm (5/8 in.) slip of West column longitudinal steel at 5.5% drift ratio.

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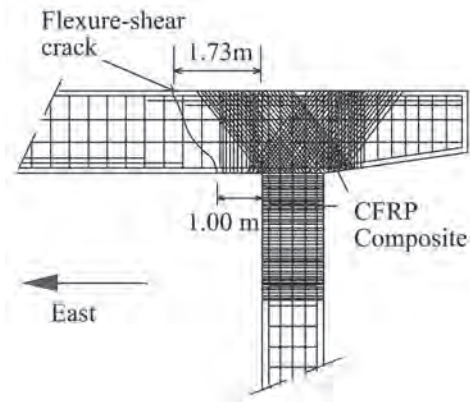


Figure 18 - Flexure-shear crack at cap beam of West column at 6% drift ratio – 2000 tests.
(1 kN = 0.225 kips, 1 mm = 0.039 in.)

Performance of an RC Corner Beam-Column Joint Severely Damaged under Bidirectional Loading and Rehabilitated with FRP Composites

by M. Engindeniz, L.F. Kahn, and A.-H. Zureick

Synopsis: This paper presents the performance of a full-scale reinforced concrete corner beam-column-slab specimen that was first severely damaged under bidirectional quasi-static loading, then rehabilitated and retested. The specimen was built using the pre-1970s construction practices including the use of low-strength materials (3000 psi [21 MPa], Grade 40 reinforcing bars) and deficiencies in reinforcement detailing. The rehabilitation process consisted of: (1) epoxy injection, (2) addition of a bar within the clear cover of the column at the inside corner, and (3) external application of a multilayer composite system made of unidirectional carbon-epoxy layers placed at different orientations. The carbon fiber-reinforced polymeric system was heat-cured at a temperature of $80^{\circ}\pm 10^{\circ}\text{C}$ ($176^{\circ}\pm 18^{\circ}\text{F}$) for 6 hours. The performance was evaluated both before and after rehabilitation based on the progression of damage and the hysteretic behavior including the changes in the strength, stiffness, and energy dissipation characteristics. The results indicated that even a severely damaged corner joint can be effectively rehabilitated using CFRP to achieve a ductile beam failure mechanism. The joint was upgraded to withstand story drift ratios of up to 3.7% applied simultaneously in both directions.

Keywords: beam-column joints; fiber-reinforced polymer; repair; strengthening

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INTRODUCTION

Reinforced concrete (RC) buildings constructed with inadequate or no seismic considerations constitute a significant portion of the building stock in many countries. Many catastrophic failures during earthquakes have indicated the vulnerability of beam-column joints in such buildings. In the U.S., buildings constructed before the adoption of the first design guidelines for RC beam-column joints in 1976 (ACI 352R-76) typically have the following major deficiencies in the joint regions: (1) strong beams, weak columns, (2) little or no joint transverse reinforcement, (3) beam bottom reinforcements with short embedment length, (4) short lapped splices of column bars just above floor level, and (5) wide spacing of column ties (Beres et al. 1996).

The ACI-ASCE Committee 352 (ACI 352R-02) recommended that the adequacy of these joints be established and that methods of improving their performance be developed. A detailed review of the technical literature shows that such efforts, in general, have been limited to testing of one-way joint specimens with no floor slab or transverse beams (Engindeniz et al. 2005). As a result, most of the proposed strengthening schemes were not only geometrically inapplicable to actual frame joints, but they also did not account for force transfer mechanisms and damage modes created by the presence of floor members and bidirectional loading. In studies where such effects were considered, the improved performance of the strengthened specimens was shadowed by the labor-intensiveness and bulkiness of the proposed strengthening schemes (e.g. concrete jacketing).

Relative advantages and disadvantages of previously studied repair and strengthening techniques were presented elsewhere (Engindeniz et al. 2005). Among these techniques, externally bonded fiber-reinforced polymeric (FRP) composite applications offer advantages over others including ease of construction, corrosion resistance, and no increase in member sizes and mass. FRP composites are most attractive for their tailorability; the fiber-orientation in each ply can be adjusted so that specific strengthening objectives such as increasing the strength only, confinement only, or both, can be achieved (ACI 440.2R-02). Tests conducted to date on joint specimens with no floor members have shown that FRP composites can be promising for achieving ductile failure mechanisms if debonding of composites from the concrete surface can be delayed or prevented. Use of FRPs in rehabilitation of actual three-dimensional frame joints, however, requires testing with consideration of the presence of floor members and bidirectional loads. In addition, thermal properties such as the glass-transition temperature and rate of curing of the matrix systems used in rehabilitation should be assessed with respect to the operating temperatures that the rehabilitated structure is likely to face during its service life. Curing schedules so adjusted may also improve the bond between the composite systems and concrete (Ferrier 1999).

RESEARCH SIGNIFICANCE

This research starts to answer “needed research” posed by ACI-ASCE Committee 352 “Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures”, and it demonstrates the efficacy of CFRP for RC beam-column joint rehabilitation.

EXPERIMENTAL PROGRAM

Specimen Design

A full-scale corner beam-column-slab specimen was designed using the pre-1970s construction practices including deficient detailing and low-strength materials (specified $f'_c=3000$ psi [21 MPa], Grade 40 reinforcing bars), and according to the ACI 318-63 code. From a review of the failure modes observed in previous studies in the literature, a column-to-beam moment strength ratio ($\Sigma M_c/\Sigma M_b$) of approximately 0.9 was targeted in the design to ensure a failure mode involving damage to the joint. Such a design called for the overall dimensions and reinforcement details shown in [Figure 1](#). The major deficiencies expected to dominate the behavior of the specimen were: (1) actual $\Sigma M_c/\Sigma M_b=0.86$, (2) no joint shear reinforcements, (3) beam bottom bars with an embedment length of only 6 in. (150 mm) into the joint, (4) a short ($20d_b$) and unconfined lapped splice of column longitudinal bars just above floor level.

Material Properties

The specimen was cast in two lifts resulting in a cold joint at the bottom of the upper column. The average compressive strength of concrete (f'_c) for the first lift was 2880 psi (19.8 MPa) on the 28th day and 3740 psi (25.8 MPa) on both test days (i.e. before and after rehabilitation). For the second lift, f'_c was 3600 psi (24.8 MPa) on the 28th day and 4950 psi (34.1 MPa) on the test days. The reinforcements consisted of #3, #5, and #6 Grade 40 deformed bars with average yield strengths of 54 ksi (372 MPa), 52 ksi (358 MPa), and 46 ksi (317 MPa), respectively.

Test Setup

The specimen represents the part of the corner of a building isolated at the assumed inflection points (i.e. midspan of beams and columns) when the building is subjected to lateral loads. The test setup is shown in [Figure 2](#). The top of the column was connected to a universal joint that allowed rotation in any direction and vertical translation; only lateral translations were restrained. The bottom of the column was fixed to the strong floor; the length of the lower column was designed such that an inflection point would form below the beam at a distance equal to the length of the upper column. Cyclic lateral loads were simulated by hydraulic actuators mounted vertically at the end of the beams. In addition to the rotational releases at each end of the actuators about an axis perpendicular to the beams, a special fixture was used between the specimen and the actuators to permit torsional rotations expected due to the presence of the slab. The column axial load was applied using a fixture consisting of a DWYDAG rod running alongside each corner of the column and a hydraulic center-hole jack connected to each rod.

Instrumentation of the As-Built Specimen

The strains in the reinforcing bars were monitored at the joint-beam, joint-column, and beam-slab interfaces in both EW and NS directions using electrical resistance strain gages mounted on the bars ([Figure 3a, b](#)). Strain gages were also mounted on two outer beam bottom bars in each direction at 3 in. (76 mm) into the joint, two stirrups above and below the joint (all four legs), and on a beam stirrup (two vertical legs) as shown in [Figure 3a](#). Joint shear strains and relative rotations of the beams and columns with respect to the joint were monitored using the LVDT configuration in [Figure 3c](#) on both S and W faces of the specimen. Additional measurement of the rotation of the beams with respect to the column was performed using string potentiometers mounted at the midwidth of the beam top and bottom surfaces ([Figure 3c](#)). Global rotations and translations of the joint region in both directions were measured with respect to a fixed reference frame using potentiometers ([Figure 3c](#)). Torsional rotations of each beam were monitored using potentiometers mounted on the inner and outer edges of the beam bottom surface at the tip of the beam and at a section near the column faces. The column shear forces were monitored by full-bridge strain gage configurations mounted on each arm of the triangular reaction frame connected to the universal joint atop the column. The column axial load was monitored by a load cell mounted on each hydraulic jack at each corner of the column.

Loading Procedure

The loading procedure simulated a cyclic lateral loading of a building carrying service gravity loads. First, a column axial load of 10% of the column's compressive load capacity was gradually applied ($0.1P_o=74$ kip [330 kN]). Then, both beams were deflected downwards in a displacement-controlled mode until the strong-axis beam moments at the column faces were brought to a level that was estimated to occur in an actual frame building under service loads. The beams were then subjected to cyclic loading symmetrically around this deformed position in a displacement-controlled mode as shown in [Figure 4](#).

The applied displacement history included unidirectional cyclic loading of EW and NS beams, respectively, at a displacement level of $\pm\Delta_y$, which was followed by bidirectional simultaneous cyclic loading of both beams at displacement levels $\pm\Delta_y$, $\pm 1.5\Delta_y$, $\pm 2.0\Delta_y$, and $\pm 4.0\Delta_y$. The objective of the unidirectional loading was to compare the behavior of the specimen to that under biaxial loading in the later cycles. The displacement at first yield, Δ_y , was determined by continuously monitoring the strains in all column and beam critical sections during the first stage of loading. The first yield occurred in the upper column at the inside (NE) corner at a displacement of $\pm\Delta_y=1.12$ in. (28.5 mm) measured from the reference deformed position used for cyclic loading. As a result, the applied displacement levels corresponded to interstory drift ratios (θ) of $\pm 0.93\%$, $\pm 1.40\%$, $\pm 1.87\%$, and $\pm 3.73\%$. Three cycles were applied at each unidirectional or bidirectional drift level to determine cyclic degradation. A displacement rate of 0.2 in./min (5.1 mm/min) was used until the beginning of the $\pm 1.87\%$ drift cycles, at which point the rate was increased to 0.3 in./min (7.6 mm/min). The loading of the as-built specimen was terminated after the $\pm 1.87\%$ drift cycles upon achieving a severe level of damage, thus preventing the specimen to become unstable under the applied loading condition. After rehabilitation, the specimen was subjected to the entire displacement history.

The following terminology is used throughout this paper: Positive direction for actuator displacements are upwards. A load step consists of moving to a certain drift level in one direction, then moving back to the reference position. A load cycle consists of a load step in the negative direction followed by another in the positive direction. A drift level that is applied simultaneously in the EW and NS directions is referred to as a bidirectional drift.

PERFORMANCE OF THE AS-BUILT SPECIMEN

Overall Behavior

The behavior of the as-built specimen was dominated by a combination of four major damage modes:

1. Yielding in the upper column during the negative load steps. The first peak at -0.93% drift in the EW direction represents the first yield accompanied by wide flexural cracks at the inside (NE) corner of the upper column. The demand at this corner increased significantly during the bidirectional cycles, the width of the cracks reaching 0.1 in. (2.5 mm) and 0.25 in. (6.4 mm) at the first peaks at -1.40% and -1.87% drift, respectively. The first yield in the lower column also occurred at the first peak at -1.87% drift.
2. Slippage/pull-out of beam bottom bars during the positive load steps. Flexural cracks at the beam-column interfaces extending vertically from the bottom to the middepth of the beams reached a maximum width of 0.06 in. (1.5 mm) during the unidirectional cycles and 0.08 in. (2.0 mm), 0.2 in. (5.1 mm), and 0.31 in. (7.9 mm) at the $+0.93\%$, $+1.40\%$, and $+1.87\%$ bidirectional drift levels, respectively. The strain readings indicated that the bottom bars near the inside faces of the beams were subjected to larger tensile forces than those on the outside. Although the bars performed well during the unidirectional cycles, and the inner bars developed strains as high as $1360 \mu\epsilon$ ($\epsilon_y=1660 \mu\epsilon$), they lost their anchorage gradually after the beginning of the bidirectional cycles. A comparison of strains just outside the joint with those at 3 in. (76 mm) into the joint indicated that, by the end of test, only the bars on the outside were left with a limited anchorage.
3. Joint shear cracking during the negative load steps. [Figure 5](#) shows that diagonal cracking in the joint occurred in one direction only and not in the often recorded "X" manner, because the loss of anchorage of the beam bottom bars prevented the formation of the other diagonal crack. The maximum width of the joint shear cracks was 0.04 in. (1.0 mm) during the unidirectional cycles. During the bidirectional cycles, they not only opened wider in the plane of the joint panel, but they also exhibited bulging in the direction perpendicular to the surface due to rotation of the hooked beam top bars. At the second peak at -0.93% bidirectional drift, these cracks had joined those at the bottom of the beam and reached a width of 0.07 in. (1.8 mm) to 0.08 in. (2.0 mm).

4. Propagation of cracks into the upper column. At the first peak at -1.40% drift, the diagonal crack in the joint panel propagated vertically into the upper column forming a combined crack extending from the bottom of the beam into the upper column, exhibiting a bulging of up to 0.25 in. (6.4 mm) in the later cycles. At the end of the test, this crack measured 0.31 in. (7.9 mm) wide at the beam embedment region, 0.25 in. (6.4 mm) wide in the joint panel, and 0.125 in. (3.2 mm) in the upper column (Figure 5a, b). The cracks in EW and NS faces were connected through the joint at an angle of roughly 45 degrees, forming a large triangular prismlike concrete block trying to separate from the outside corner of the column. The two stirrups along the column lapped splice resisted this separation by developing strains as high as 1550 $\mu\epsilon$ in their S and W legs.

The maximum tensile strains achieved in the beam top bars provide a means of comparing the specimen's performance with a desirable ductile beam hinging mechanism. The strain measurements at the NS beam-column interface showed that while the outermost top bar developed 90% of its yield strain during the bidirectional cycles, the innermost top bar developed only 42%. A similar strain distribution was recorded at the EW beam-column interface. Such a strain distribution is attributed to combined unsymmetric bending and torsion in the beams due to the presence of the slab. The measured strain levels indicated that a significant strengthening of the column and the joint was needed to achieve yielding in all beam top bars.

Two major cracks were observed in the slab:

1. A flexural crack that formed in a direction parallel to the NS beam at 3.5 in. (89 mm) from the inside face of this beam and propagated across the entire width and through the thickness of the slab—This crack initiated during the EW unidirectional cycles and reached a maximum width of 0.02 in. (0.5 mm). Such a crack did not occur in the EW direction during the NS unidirectional cycles.
2. A diagonal crack at the bottom surface extending from the joint region to the opposite corner—This crack was located after completing the test and measured 0.04 in. (1.0 mm)-wide. It indicated the need for special diagonal reinforcement in the slab corners in line with the requirements of ACI 318-05, Section 13.3.6.

Hysteretic Behavior

The strength, stiffness, and energy dissipation characteristics of the specimen were evaluated through the force-drift hysteretic response, presented in Figure 6 for both EW and NS directions. The curves are shifted from the horizontal line representing zero load due to the initial loads on the actuators (EW=2.0 kip [8.9 kN], NS=2.7 kip [12.0 kN]) at the deformed position used as the reference level for cyclic loading. This deformed position is the “zero” drift datum. Positive load values indicate compression force in the hydraulic ram.

The peak loads measured in each cycle in both negative and positive loading directions are presented in Table 1. The maximum loads in both positive and negative directions occurred in the first unidirectional cycles. A significant decrease in the peak loads was observed at the beginning of the bidirectional cycles, then the strength degraded more gradually. Both the overall and cyclic strength degradation were more pronounced in the negative direction as a result of the progressive damage in the joint panel and in the upper column.

Significant degradation in stiffness and pinching of the curves were dominant throughout the hysteresis loops. The peak-to-peak stiffness K_p , defined as the slope of the line connecting the peak of a negative load step to that of the next positive load step, decreased continuously as shown in Figure 7. An 80% decrease in K_p from the first cycle to the last cycle was observed in the EW direction; the decrease in K_p in the NS direction was 78%. The degradation in stiffness was more significant after the beginning of the bidirectional cycles.

The energy dissipated by the specimen was calculated as the area enclosed by the force-displacement curves. Cumulative plots of the dissipated energy in both EW and NS directions are presented in Figure 8. Although the total energy increased continuously, the increase was not proportional to the increase in the applied displacements due to the combined effects of increased pinching and strength degradation.

The plot of the joint shear stress factor (γ) versus the joint shear strain (γ_s) is presented in Figure 9, where $\gamma = V_{jh} / \sqrt{f'_c b_j h_c}$ as defined in ACI 352R-02, Sec. 4.3.1. The horizontal joint shear force (V_{jh}) was determined as the difference between the column shear force and the total tensile normal force (T) acting on the beam-joint interface. T was estimated by assuming a constant moment-arm of $jd=0.875d$ between the tension and compression resultants at this section (i.e. $T=M/0.875d$). γ_s for each joint panel was taken as the average of four joint shear strain estimates obtained from the LVDT configuration in Figure 3c. The resulting γ versus γ_s plots were in line with all

previous discussions in that significant softening and distortion of the joint during the negative load steps was clear. Maximum shear stresses were in the negative direction, the γ factor being $5.7\sqrt{\text{psi}}$ ($0.47\sqrt{\text{MPa}}$) and $4.9\sqrt{\text{psi}}$ ($0.41\sqrt{\text{MPa}}$) for the EW and NS directions, respectively. For the positive loading direction, maximum γ values of $4.2\sqrt{\text{psi}}$ ($0.35\sqrt{\text{MPa}}$) and $4.5\sqrt{\text{psi}}$ ($0.37\sqrt{\text{MPa}}$) were obtained for the EW and NS directions.

The shear capacity of the joint may be underestimated by the use of the “constant moment-arm approximation” because of the unaccounted contributions to the tensile forces in the beam bars. Data not included here for brevity suggests that torsion, for instance, increases the tensile forces in all the beam bars. Even so, this method can be used to compare the joint shear strength before and after rehabilitation.

REHABILITATION PROCEDURE

All rehabilitation steps were performed at the deformed position used as reference for cyclic loading. The specimen was first repaired by pressure-injection of a high-strength, high-modulus, low-viscosity epoxy that filled all cracks larger than 0.01 in. (0.3 mm) [Figure 10a]. The injection of approximately 2 gal. (7.6 L) of epoxy confirmed the severity of damage in the specimen.

Strengthening Design and Application

The specimen was strengthened to achieve improvements in the following areas: (1) flexural strength of the column, (2) joint shear strength, (3) beam bottom bar anchorage, (4) column end confinement, and (5) flexural strength at the SW corner of the slab. Strengthening in all areas except (1) was performed using a 9 oz/yd² (300 g/m²) unidirectional carbon fabric impregnated with a high-strength, high-modulus epoxy matrix using a hand-layup technique. During the preliminary design of the experiment, tensile properties of the carbon-epoxy system were taken as those reported by the manufacturer (i.e. tensile strength $P_u^{CFRP}=2100$ lb/in./layer [370 N/mm/layer], ultimate strain $\epsilon_u^{CFRP}=1.00\%$, thickness $t=0.020$ in./layer [0.5 mm/layer]). The properties attained in the actual application were examined by testing coupons cut from a single-layer plate (i.e. witness plate) made using the same technique and curing schedule (see next section) as that used in rehabilitating the specimen. The witness plate contained 42% fibers by weight, which corresponded to a fiber volume ratio of approximately 34% assuming a 4% void volume in the system. Based on 10 coupon tests, the following average values were obtained: $P_u^{CFRP}=2930$ lb/in./layer (510 N/mm/layer), $\epsilon_u^{CFRP}=1.52\%$, and $t=0.027$ in./layer (0.7 mm/layer).

The following approach was taken in the design and application of strengthening in each area:

- 1. Flexural strength of the column.** The higher force demand on the inside corner of the upper column required addition of flexural reinforcement along the entire height of the column and continuous through the slab. The amount of existing column longitudinal reinforcement (2#5) at this corner was doubled by removing a 2 in. x 2 in. (50 mm x 50 mm) portion of the column corner including a perforation in the slab, adding a #7 bar ($f_y=65$ ksi [448 MPa]) (Figure 10c), and recasting the corner with a polymer-modified, cementitious mortar ($f'_c=7200$ psi [49.6 MPa] on the test day).
- 2. Joint shear strength.** The maximum horizontal joint shear force (V_{jh}^{max}) that could develop was estimated by assuming yielding of all beam top bars and the three slab bars within 20 in. (510 mm) (ACI 318-05, Sec. 8.10.3) next to the beams:

$$V_{jh}^{max} = \sum A_s f_y - V_{col} = 6(A_s f_y)_{\#6} + 3(A_s f_y)_{\#3} - \frac{M_n^-}{L_b} 1.25 \quad (1)$$

where M_n^- is the negative moment capacity of the beam, L_b is the beam length measured from column face, and 1.25 is the ratio of the beam end load to column shear (V_{col}) analytically determined for this test setup using the measured material properties. Three layers of CFRP oriented perpendicular to the longitudinal axis of the column (90 degrees) were required to resist this force according to Equation 2:

$$n_{jh} = \frac{V_{jh}^{max}}{h_j P_u^{CFRP}} \quad (2)$$

where n_{jh} is the number of layers, h_j is the height of the joint covered with CFRP (20 in. [510 mm]), and P_u^{CFRP} is the average tensile strength (lb/in./layer) of CFRP ($n_{jh}=2.62=3$). An additional two layers oriented at 0 degree (parallel to the column) were provided to resist the vertical component of the principal tension force in

the joint panel. All layers were extended into the upper and lower columns by a distance equal to the height of the confined regions at the column ends. All five layers were applied on the S and W faces of the specimen only.

3. **Beam bottom bar anchorage.** Although the demand was larger on the bottom bars near the inside faces of the beams than on those near the outside faces, for ease of construction, an attempt was made to improve the positive moment capacity of the beams by providing CFRP reinforcement on the outside faces only. CFRP strips were placed around the SW corner of the joint such that the strips extended 28 in. (710 mm) on to the beams measured from the beam-column interfaces (“Area 4” in Figure 10g). The number of layers (n_{strip}) was selected such that the load capacity of the strips would be equivalent to that of all three bottom bars at yield:

$$n_{strip} = \frac{\sum A_s f_y}{w_s P_u^{CFRP}} = \frac{3(A_s f_y)_{\#6}}{w_s P_u^{CFRP}} \quad (3)$$

The width of the strips (w_s), preferred to be less than half the beam depth to avoid an undesirable increase in the negative moment capacity of the beams, was arbitrarily selected as 7 in. (180 mm); this resulted in $n_{strip} = 4.13 = 5$. An anchorage length of 28 in. (710 mm) was selected so that the distance from the tip of the beam bottom bars to the end of the strips would be equivalent to the development length of these bars according to ACI 318-05, Sec.12.15.

4. **Column end confinement.** The CFRP used for confinement of the column ends was designed per ACI 440.2R-02, Sec. 11.3 such that the effective usable compressive strain in concrete would be equivalent to that provided by the hoop reinforcement required by ACI 318-05, Sec. 21.4.4. This approach called for two layers (90 degrees) of wrapping at the column ends over a length of 18 in. (460 mm) [“Area 1” in Figure 10g]. Wrapping the column ends also provides an effective means of anchorage for the layers extended from the joint onto the columns as well as for the column longitudinal bar added in the NE corner.
5. **SW corner of the slab.** A 48 in. x 48 in. (1220 mm x 1220 mm) triangular portion of the bottom surface was strengthened with two layers of CFRP to increase the flexural strength to an extent required by ACI 318-05, Sec. 13.3.6. The fibers in both layers were oriented in the NW-SE direction (“Area 5” in Figure 10h). Strengthening at the top surface was deemed unnecessary due to limited cracking in this region.

It is to be noted that the use of the manufacturer’s reported nominal CFRP properties in the design and rounding up of the resulting numbers of required layers (e.g. n_{jh} , n_{strip}) to whole numbers lead to the use of a few more layers of CFRP than what would be used if the properties attained in the actual application were known. For instance, the as-attained properties would lead to $n_{jh}=2$ (instead of 3) and $n_{strip}=3$ (instead of 5); the efficacy of such a configuration is to be studied in subsequent experiments.

Curing of CFRP

Our differential scanning calorimetry (DSC) tests conducted on the matrix system used for the FRP system showed that room temperature-cured Sikadur 330 US epoxy had glass transition temperature (T_g) values of 49°C (122°F) and 56°C (133°F) after 9 and 60 days, respectively. This raised a concern about the mobility of the epoxy molecular chain structure in an operating temperature range that is often encountered in the vast majority of regions around the world. It is often recommended that the FRP system have a T_g of 10°C (18°F) to 30°C (54°F) higher than the operating temperature (e.g. Zureick and Kahn 2001; MIL-17 2002; AFGC 2003).

To achieve an acceptable degree of molecular cross-linking, the matrix system was heat-cured at 80°C (176°F) for a duration of 3 hours or more. DSC results indicated that, after 3 hours, the T_g was increased to 70°C (158°F). Subsequently, all areas of the specimen that were strengthened with CFRP were heat-cured at a temperature of 80°±10°C (176°±18°F) for 6 hours instead of 3 hours, an arbitrary decision that was based on the fact that part of the heat would be absorbed by concrete. Heat-curing was performed by building a sealed, insulated wooden box around the region, and then heating the enclosed region with four adjustable heat guns (Figure 10f). DSC tests conducted on samples taken from the FRP system after the completion of the test program resulted in T_g values between 63°C (145°F) and 65°C (149°F).

Layup Sequence

The CFRP layers were stacked in a sequence that would result in a symmetric layup in all areas. A symmetric layup was necessary to: (1) prevent curvatures during heat-curing that could cause early debonding from the

concrete surface, and (2) eliminate the extension-bending coupling that would result in difficulties associated with the determination of the tensile properties of the FRP system. The final stacking sequences in all areas are shown in Figure 10g, h.

Instrumentation of the Rehabilitation Components

The strains in the added #7 bar at sections above and below the beams were monitored using two strain gages at each section (Figure 3e). The strains in the FRP strips bonded on each beam were monitored using three strain gages mounted on the strips at a section 4 in. (102 mm) away from the beam-joint interface (Figure 3d). These gages were evenly spaced with the bottom one being at the level of the beam bottom bars. Strain gages were also mounted on the column end wraps at the midwidth of the column at the first and second stirrup levels above the joint and at the third stirrup level below the joint (Figure 3d).

PERFORMANCE AFTER REHABILITATION

Overall Behavior

After rehabilitation, the poor behavior of the as-built specimen was improved to a desirable strong column-weak beam behavior. The initiation of damage was delayed until the beginning of the bidirectional cycles, and the damage modes leading to the failure of the specimen started to occur only after the $\pm 1.87\%$ drift cycles. Improvements in performance are summarized below in reference to the damage modes that dominated the behavior of the specimen before rehabilitation:

1. **Onset and progression of bar yielding.** The first yield was delayed until the first peak at -0.93% bidirectional drift at which point yielding occurred in both beams and in the slab. The second beam top bars from outside in both beams and a slab top bar parallel to the NS near the column yielded, the outermost beam top bars just reached their yield strain ($\epsilon_y = 1660 \mu\epsilon$), and the strains in the other beam top bars ranged from $1230 \mu\epsilon$ to $1330 \mu\epsilon$. The flexural cracks near the top of both beams measured 0.02 in. (0.5 mm) wide. At the first peak at -1.40% drift, the two outer beam top bars in each beam and two slab top bars near the column parallel to each beam had yielded; the other beam top bars had just reached their yield strain. The flexural cracks near the top of the beams measured 0.07 in. (1.8 mm) and 0.05 in. (1.3 mm) wide for the EW and NS beams. The #7 bar added in the NE corner of the column reached its yield strain ($\epsilon_y = 2490 \mu\epsilon$) in the upper column at the first peak at -1.87% drift at which point all beam top bars except for the innermost NS beam top bar had yielded, and the cracks in the EW and NS beams were 0.15 in. (3.8 mm) and 0.08 in. (2.0 mm) wide. During the $\pm 1.87\%$ and $\pm 3.73\%$ drift cycles, the EW beam underwent more damage than the NS beam, and the strains in the beam bars increased more rapidly than in the column bars as drift increased. At the first peak at -3.73% drift, the EW beam underwent severe spalling. The innermost NS beam top bar did not yield throughout the test but developed strains as high as $1490 \mu\epsilon$. While the NS beam top bars that did yield developed strains as high as $14,080 \mu\epsilon$, the demand on the column was limited to a maximum strain of $5250 \mu\epsilon$ in the added #7 bar in the upper column and spalling of the inside corner of the joint near the first negative peak at -3.73% drift.
2. **Beam bottom bar anchorage.** The FRP strips bonded on the beams ("4" in Figure 10g) prevented the distress from concentrating at the column face. At the third peak at $+0.93\%$ bidirectional drift, distress consisted of 0.01 in. (0.3 mm)-wide flexural cracks at the bottom surfaces of the beams distributed over a distance of 50 in. (1270 mm) from the column face, and cracks at the beam-column interface measuring 0.01 in. (0.3 mm)-wide under the FRP strips and 0.04 in. (1.0 mm)-wide near the inside corner of the column. At the first peak at $+1.40\%$ drift, a complete debonding of the FRP strips on the NS beam took place suddenly, initiating at the beam-column interface and moving along the beam. The FRP strips developed a maximum strain of $4570 \mu\epsilon$ at the level of the beam bottom bars. The strips on the EW beam were debonded over a distance of 10 in. (250 mm) from the beam-column interface, and complete debonding did not occur until the first peak at $+1.87\%$ drift. A maximum strain of $3920 \mu\epsilon$ was developed in these EW strips. After debonding of the FRP strips, the beams lost much of their improved behavior in positive bending. A thin layer of concrete attached to the debonded strips indicated the need for better anchorage schemes for these strips so the resulting interfacial and normal stresses can be maintained by the concrete.
3. **Joint shear capacity.** The problem of joint shear failures prior to ductile failure of the beams was resolved. Although very small portions of the FRP system in the joint ("2" and "3" in Figure 10g) started to debond near

the beam-joint interfaces during the $\pm 1.87\%$ drift cycles, the failure of the joint required a drift of -3.73% , at which point the beams had undergone significant yielding and damage. The failure of the joint involved crushing of the joint core and separation of the FRP system with large pieces of the joint concrete attached to it. This indicated a remarkable performance of the FRP system in maintaining the bond to concrete until the failure of the joint concrete.

4. **Column end confinement.** No significant signs of damage were observed at the column ends throughout the test. The tendency of the upper column to expand just above the joint was prevented by the FRP wrap which developed strains as high as $9730 \mu\epsilon$ and $6880 \mu\epsilon$ at the first stirrup level in the W and S faces, respectively, at the third peak at -3.73% drift. At this point, the W leg of the first stirrup above the joint had just yielded ($\epsilon_y = 1960 \mu\epsilon$), and the other legs had developed strains up to $1720 \mu\epsilon$. The strains measured at the second stirrup level 11.5 in. (290 mm) above the slab indicated that there was no tendency of the column to expand.

The epoxy-injected cracks in the slab did not reopen but new cracks formed next to them and at other locations. At the bottom surface, a diagonal crack with a maximum width of 0.05 in. (1.3 mm) formed between the edge of the triangular FRP-strengthened region and the outer corner, which was accompanied by two other cracks extending from the corners of the triangular region in a direction parallel to the diagonal. At the top surface, flexural cracks formed near both EW and NS beam-slab interfaces across the entire width of the slab (width ≤ 0.03 in. [0.8 mm]). Additional flexural cracks formed in both directions away from the beams with widths up to 0.02 in. (0.5 mm).

Hysteretic Behavior

The hysteretic behavior of the specimen before and after rehabilitation are compared in [Figure 6](#) through [Figure 9](#) and in [Table 1](#).

The strong column-weak beam behavior manifested itself in the force-drift hysteresis loops ([Figure 6](#)) as increases in the negative peak loads that were maintained until the first peak at -3.73% drift, after which deflection the peak loads decreased due to significant damage in the beams. The maximum negative loads were increased by 57% and 70% for the EW and NS beams, respectively, when compared with those obtained for the as-built specimen. For the positive loading direction, up to 40% and 27% increases in the maximum loads were achieved for the EW and NS beams, respectively, until the FRP strips on the side of the beams debonded. Debonding of the strips are marked “7b” and “10b” on the hysteretic plots in [Figure 6](#). Percent increases in the peak loads were calculated with respect to the initial loads on the actuators at the deformed position used as reference for cyclic loading, represented by horizontal dashed lines in [Figure 6](#).

The increases in peak-to-peak stiffness (K_p) during the bidirectional cycles ranged from 82 to 160% for the EW direction and from 68 to 117% for the NS direction ([Figure 7](#)). Unlike in the as-built specimen, much of the initial stiffness was maintained through the bidirectional cycles especially until bedonding of the FRP strips on the beams.

The increase in the amount of energy dissipated by the specimen after the onset of yielding in the beams is clearly observed in [Figure 8](#). Until the end of the $\pm 1.87\%$ drift cycles, where loading of the as-built specimen was terminated, the rehabilitated specimen dissipated 39% and 37% more energy in the EW and NS directions, respectively. During the $\pm 3.73\%$ drift cycles, the amount of the dissipated energy was doubled; a majority of this energy was dissipated during the first cycle where a complete ductile failure of the beams was achieved.

The shear stress-strain behavior of the joint was improved significantly toward a desirable “rigid joint” behavior ([Figure 9](#)). Notable distortion of the joint occurred only after crushing of the joint concrete at the first peak at -3.73% drift. The maximum joint shear stress factors (γ) of $8.3\sqrt{\text{psi}}$ ($0.69\sqrt{\text{MPa}}$) and $8.0\sqrt{\text{psi}}$ ($0.66\sqrt{\text{MPa}}$) obtained for the negative loading direction for the EW and NS directions represent increases of 46% and 63% over the values obtained for the as-built specimen. As previously discussed, the actual shear strength of the joint may be underestimated by the “constant moment-arm approximation” used in determining the reinforcing bar tensile forces at the beam-joint interfaces. For example, another estimate of γ can be obtained for the load state where all the EW beam top bars and two slab top bars near this beam have yielded. Assuming that the second layer of beam top reinforcements have also yielded (not instrumented), and not considering any strain hardening, a γ value of approximately $10.8\sqrt{\text{psi}}$ ($0.90\sqrt{\text{MPa}}$) was obtained. Such a γ value is very close to that recommended by the ACI 352R-02 for use in the design of new corner joints that have members required to dissipate energy through reversals of deformation into the inelastic range (i.e. Type 2 joints: $\gamma = 12\sqrt{\text{psi}}$ [$1.00\sqrt{\text{MPa}}$]).

CONCLUSIONS

The following conclusions were drawn from the experimental results presented in this paper:

1. The behavior of a nonseismically designed reinforced concrete corner beam-column joint was dominated by a combination of column bar yielding, joint shear failure, and loss of anchorage of beam bottom bars.
2. Consideration of bidirectional loading and presence of all floor members revealed damage modes that may be overlooked in two-dimensional tests and analyses (e.g. larger demands on the inside corner of the column and on the beam bottom bars near the inside faces of the beams).
3. Even a severely damaged corner joint was efficiently rehabilitated to meet the strong column-weak beam criterion through easy-to-implement procedures such as the addition of a reinforcing bar within the clear cover of the column and the bonding of a CFRP system that was heat-cured in place.
4. Development of the positive moment capacity of the beam needs improvement. CFRP strips bonded on the outside faces of the beams were effective in anchoring the beam bottom bars, and they developed the yield stress in the exterior bar in the EW beam. But the full ductile capacities of the exterior bars were not developed, and the interior bottom bars were not brought to yield. Improvement in the anchorage of these CFRP strips and/or additional strengthening near the interior beam bottom bars may be needed.
5. Heat-curing of the composite system in place provided an effective means for achieving a full cure of the epoxy matrix quickly and for increasing the glass-transition temperature to values above the maximum operating temperature. While a superior composite-concrete bond was also achieved for the current specimen, additional data are necessary to evaluate the contribution of heat-curing to this performance.

The scope of this paper was limited to discussion of the global behavior of the specimen. A more detailed investigation of the internal force transfer mechanisms that lead to this global behavior will be the subject of a future publication.

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Table 1 — Peak Loads

EW						NS			
Before Rehab				After Rehab		Before Rehab		After Rehab	
Drift, θ	Cycle	$-P_{max}$ (kip)	$+P_{max}$ (kip)	$-P_{max}$ (kip)	$+P_{max}$ (kip)	$-P_{max}$ (kip)	$+P_{max}$ (kip)	$-P_{max}$ (kip)	$+P_{max}$ (kip)
EW $\pm 0.93\%$	1	-5.83	10.64	-9.11	13.73	*		*	
	2	-4.97	10.28	-8.68	13.42				
	3	-4.62	9.98	-8.43	13.22				
NS $\pm 0.93\%$	1	*		*		-4.03	11.40	-7.13	13.75
	2					-3.74	10.91	-7.20	13.33
	3					-3.46	10.64	-6.98	13.08
Biaxial $\pm 0.93\%$	1	-2.35	8.79	-7.25	12.93	-1.84	9.40	-6.24	12.56
	2	-1.75	8.51	-6.89	12.54	-1.21	9.04	-5.87	12.20
	3	-1.35	8.29	-6.63	12.39	-0.85	8.80	-5.60	12.03
Biaxial $\pm 1.40\%$	1	-2.46	8.31	-10.23	14.08	-1.91	9.02	-8.77	10.88
	2	-1.13	7.72	-8.86	13.39	-0.97	8.44	-7.65	10.20
	3	-0.61	7.45	-8.00	12.99	-0.42	8.14	-6.84	9.85
Biaxial $\pm 1.87\%$	1	-1.45	7.86	-9.86	11.11	-1.06	8.53	-8.15	10.23
	2	-0.10	7.52	-7.97	10.35	-0.02	8.01	-6.55	9.75
	3	0.28	7.30	-6.99	9.92	0.66	7.71	-5.55	9.46
Biaxial $\pm 3.73\%$	1	**		-9.63	9.43	**		-6.83	10.41
	2			-4.88	8.58			-2.71	9.32
	3			-2.31	8.21			-1.08	***

* Peak loads during the unidirectional cycles are reported for the direction of loading only.

** Testing of the as-built specimen was terminated after the $\pm 1.87\%$ bidirectional drift cycles.

*** $+P_{max}$ in this cycle could not be recorded due to a data acquisition problem.

1 kip = 4.448 kN

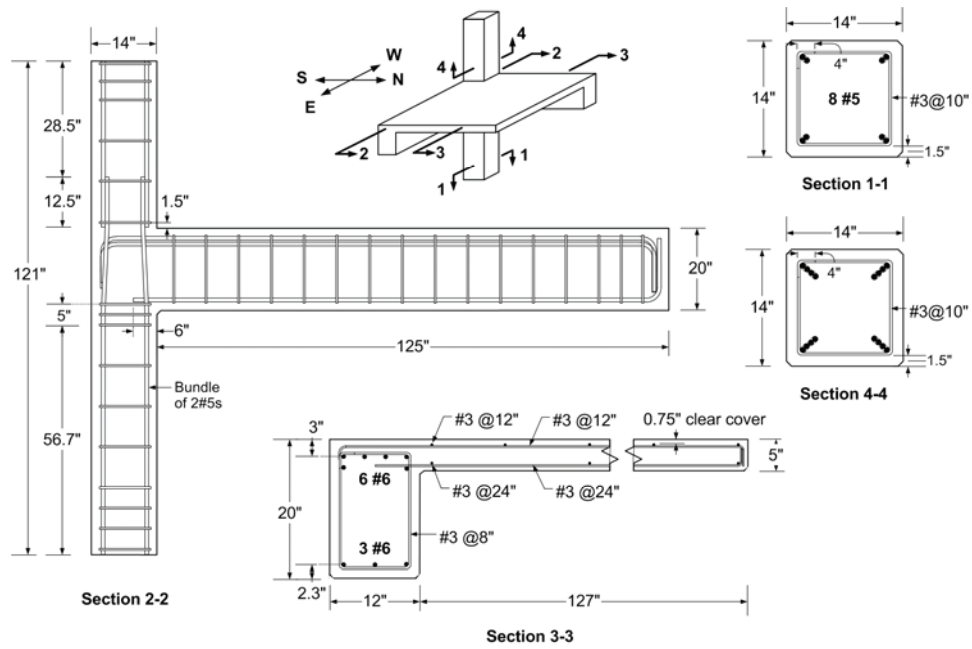


Figure 1 — Reinforcement details (1 in. = 25.4 mm).

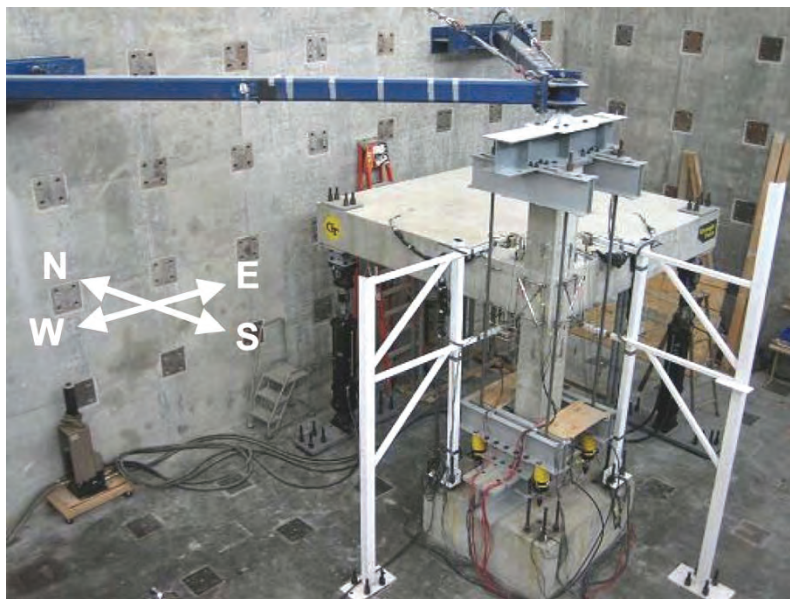


Figure 2 — Test setup.

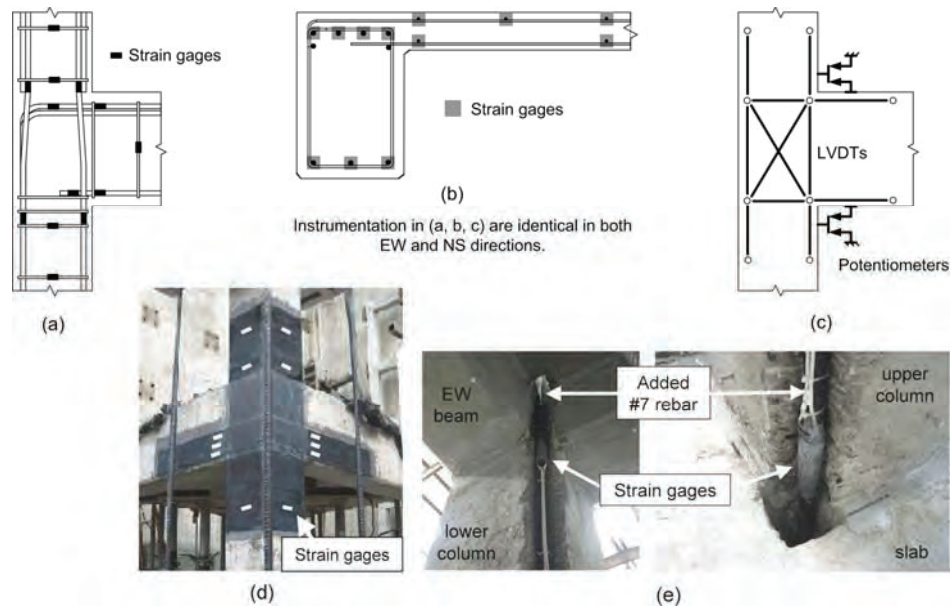


Figure 3 — Instrumentation.

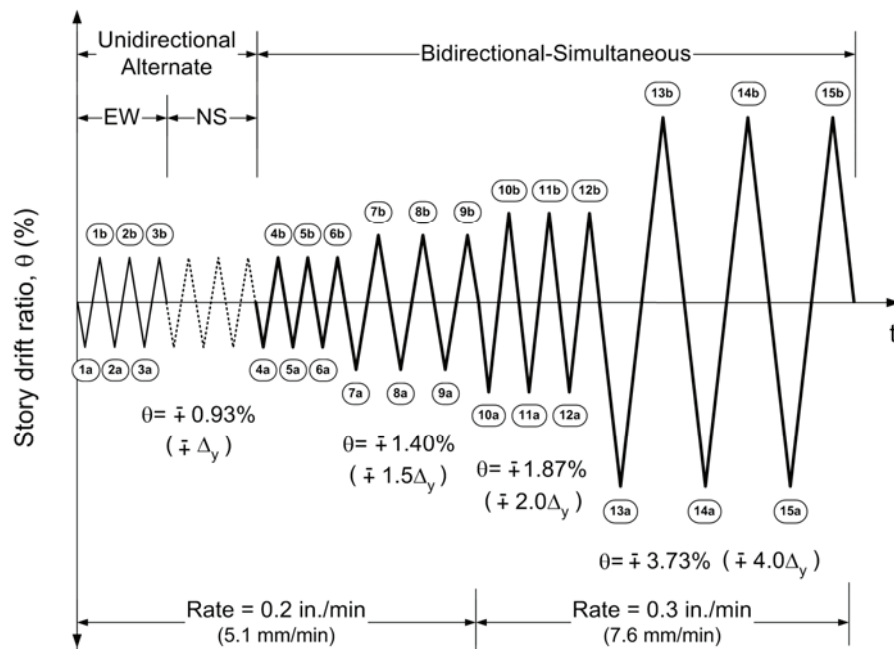


Figure 4 — Applied displacement history. Negative drift (downward beam displacement) causes tension at the top of the beams. Cycle 1 has negative peak 1a and positive peak 1b.



(a) West face



(b) South face

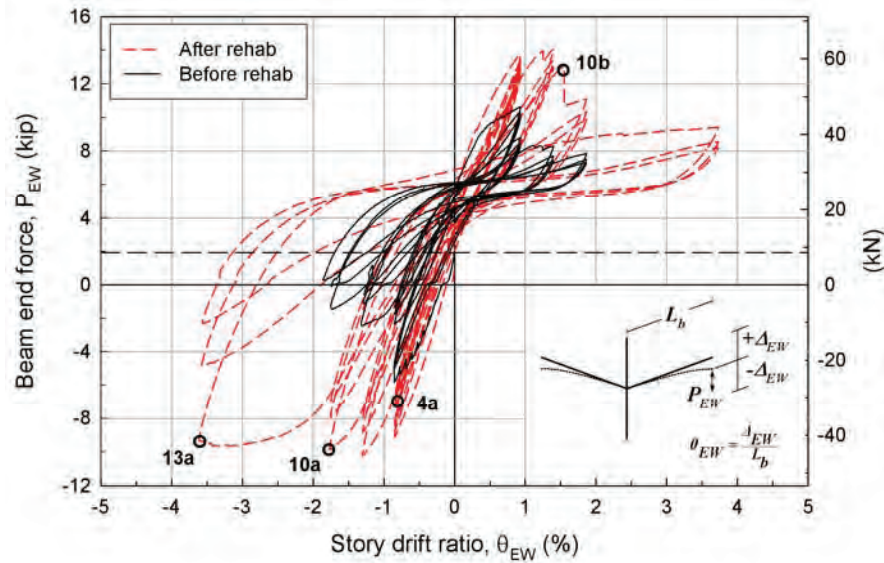


(c) West face

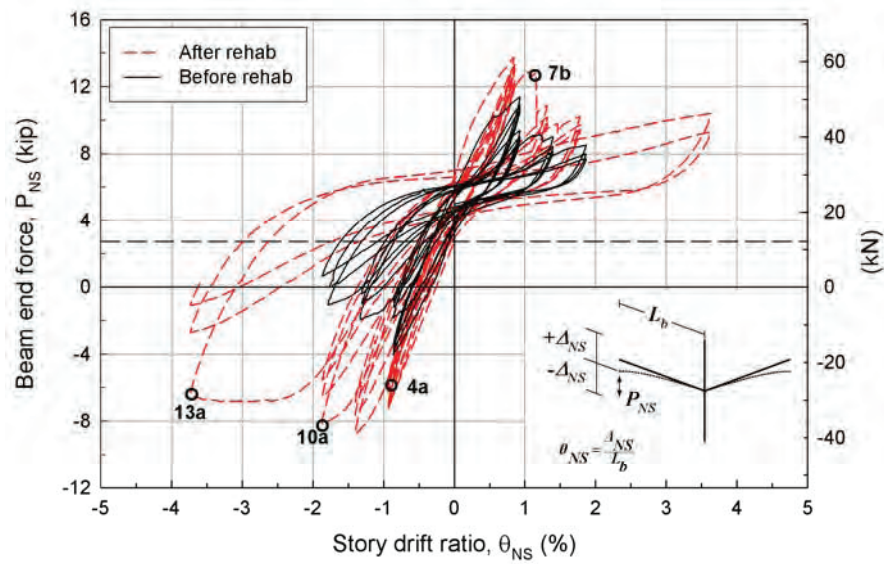


(d) South face

Figure 5 — Damage at the end of test: (a, b) before rehabilitation at cycle point 12a, and (c, d) after rehabilitation at cycle point 15a.



(a)



(b)

(4a) First yield in beams, (7b) Debonding of FRP strips on NS beam,
(10a) Extensive yielding in beams and first yield in column,
(10b) Debonding of FRP strips on EW beam, (13a) Joint failure

Figure 6 — Force-drift hysteresis loops: (a) EW, and (b) NS.

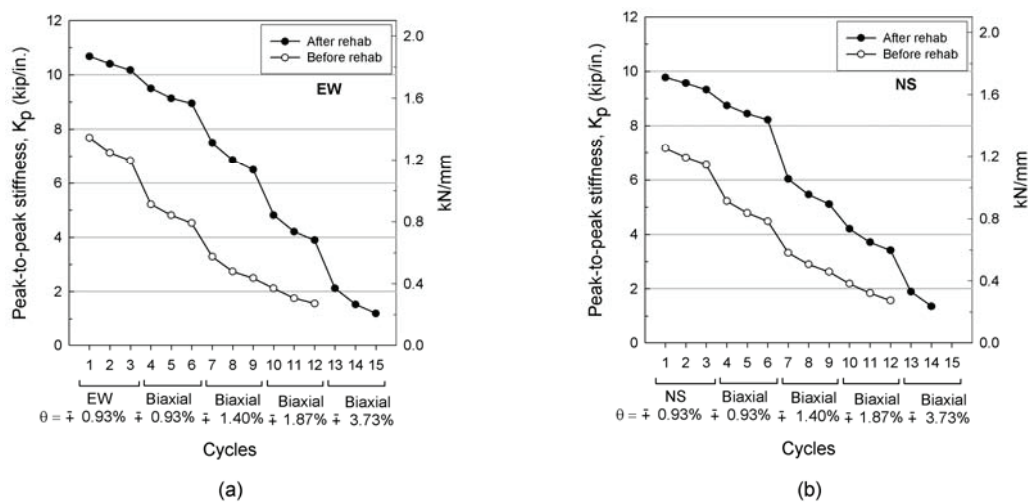


Figure 7 — Stiffness degradation: (a) EW, and (b) NS.

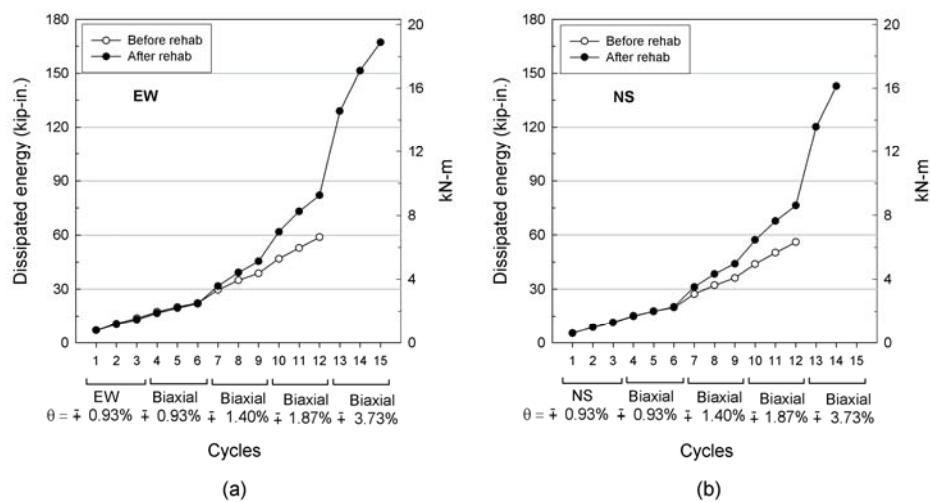


Figure 8 — Cumulative dissipated energy: (a) EW, and (b) NS.

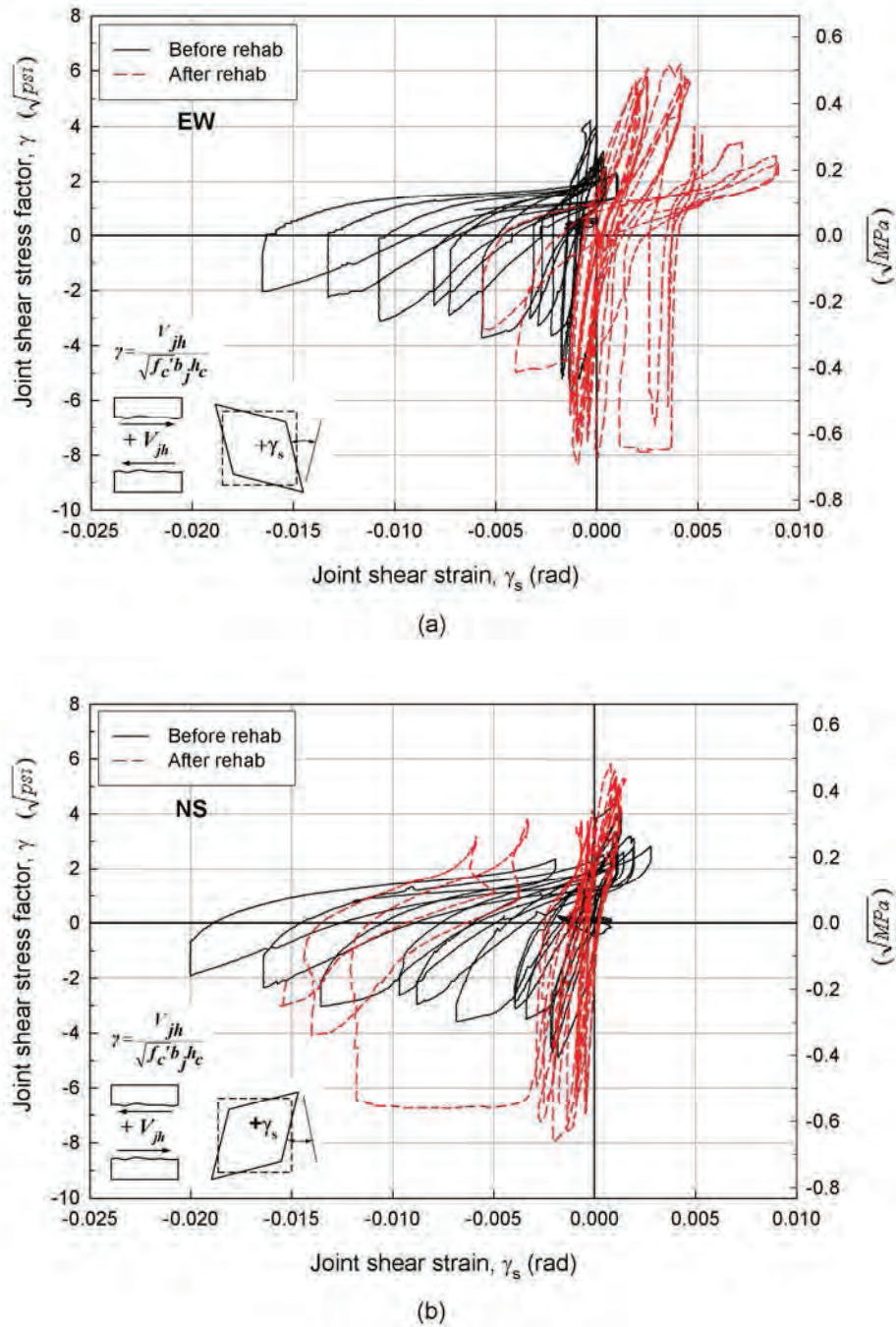


Figure 9 — Joint shear stress-strain hysteresis loops: (a) EW, and (b) NS.

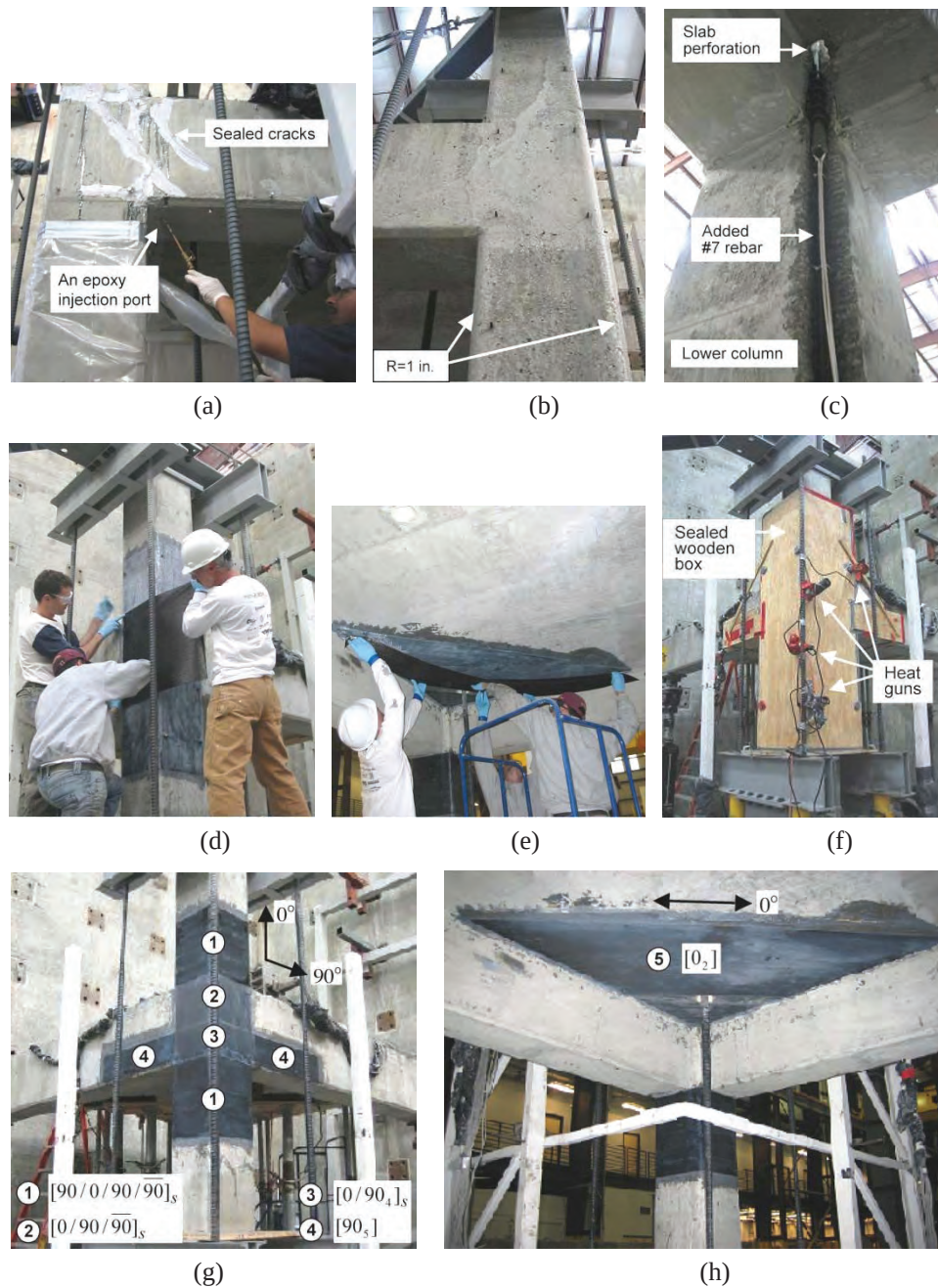


Figure 10 — Rehabilitation process: (a, b) epoxy injection and surface preparation, (c) addition of a #7 reinforcing bar, column inside corner, (d, e) CFRP application on column and bottom of slab, (f) heat-curing of CFRP, and (g, h) finished look.

Experimental Evaluation of Noncircular Reinforced Concrete Columns Strengthened with CFRP

by S. Rocca, N. Galati, and A. Nanni

Synopsis: This paper presents the results of an experimental investigation on the axial behavior of medium and large scale Reinforced Concrete (RC) columns of circular and non-circular cross-sections strengthened with unidirectional Carbon Fiber Reinforced Polymer (CFRP) wraps. A test matrix was developed to investigate the effect of different variables, such as the geometry of the specimen cross-section (circular, square, and rectangular), the side aspect ratio, and the area aspect ratio. A total of 22 specimens were divided into six series of three specimens each and two series of two specimens each. The largest and smallest columns featured cross-sectional areas of 0.8 m^2 (9 ft^2) and 0.1 m^2 (1 ft^2), respectively. All the specimens were subjected to pure axial compressive loading. The experimental results are compared with available data on RC specimens with one minimum dimension of the cross-section of 300 mm (12 in.). This evaluation allowed confirming that among circular and non-circular specimens of the same cross-sectional area and FRP volumetric ratio, the level of confinement effectiveness decreases as the side aspect ratio increases. Additionally, size effect within specimens of circular cross-section does not appear to be significant; however, for the case of non-circular specimens, scatter and limitation of data-points does not allow at the present time to draw a definite conclusion. A new analytical method that allowed estimating the confining pressure in non-circular cross-sections from the transverse strains at the corners is proposed. The obtained confining pressures and experimental results from this study allowed calibrating a strength model, which was validated with the available experimental data in the literature. Finally, the predictions of this strength model were compared to the ones by the model of Lam and Teng yielding close agreement.

Keywords: confinement; ductility; FRP strengthening; noncircular columns; reinforced concrete

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INTRODUCTION

Confinement of Reinforced Concrete (RC) columns by means of Fiber Reinforced Polymer (FRP) jackets is a technique being used with growing frequency to seek the increment of load carrying capacity and/or ductility of such compression members.

The confinement of non-circular columns is generally acknowledged to be less efficient than the confinement of circular columns, since in the latter case, the wrapping provides circumferentially uniform confining pressure to the radial expansion of the compression member. In non-circular columns, the confinement is concentrated at the corners rather than over the entire perimeter.

Extensive work in both the experimental and analytical areas has been conducted on small-scale plain concrete specimens of circular and non-circular cross-sections confined with FRP and subjected to pure axial compressive loading (Carey and Harries 2003; De Lorenzis and Tepfers 2003; Lam and Teng 2003a,b; Masia et al. 2004). Studies focused on RC columns of both circular and non-circular cross-sections of considerable size (minimum-dimension of 300 mm [12 in.]) have also been conducted (Demers and Neale 1994; Kestner et al. 1997; Wang and Restrepo 2001; Youssef 2003; Carey and Harries 2003; Matthys et al. 2005); however, this experimental research has been limited due to high cost and lack of high-capacity testing equipment. This situation has been the main reason for overlooking the following important effects on the element performance that have not been accounted for in most of the available models: (a) the size of the cross-sectional area; (b) the dimensional aspect ratio of the cross-sectional area; (c) the possible detrimental effect of longitudinal steel reinforcement instability; (d) the concrete dilation dependant on the volumetric ratio; (e) and the contribution of the internal transverse steel reinforcement. In spite of these obstacles, several models to predict strength enhancement and in some instances stress-strain behavior have been proposed (Wang and Restrepo 2001; Lam and Teng 2003b; Maalej et al. 2003) and have become the basis for design provisions.

RESEARCH SIGNIFICANCE

So far, the vast majority of tests on non-circular columns have been on small samples. However, the design methods based upon these tests may not be reliable in predicting the strength enhancement and ductility in terms of axial deformation that might be achieved for larger columns found in practice. A systematic experimental investigation of the effect of column size is presented herein. This research study is now limited to specimens under pure axial loading condition, and this is considered the first step to understand the confinement process. Future work should include the effect of flexure and shear in order to develop complete interaction diagrams.

This research is of practical relevance in that there are thousands of RC structures (bridges and buildings) having non-circular columns that due to increases in load demands, changes in use or additions, or code updates require rapid and efficient strengthening with minimum disruption to users. Wrapping non-circular columns with FRP has the potential to achieve increments in strength and ductility with ease of installation, provided that fundamental behavior is understood.

EXPERIMENTAL PROGRAM

Test Program

The test matrix ([Table 1](#)) was developed to investigate the influence the effects of mainly two variables: the side aspect ratio (h/b) and area aspect ratio $A_g/A_{g(C)}$ on the effectiveness of FRP confined RC columns of non-circular cross-sections. The test matrix was divided into two sub-matrices based on the laboratories where the experiments were carried out: CALTRANS Seismic Response Modification Device Testing Laboratory (SRMD) at the University of California San Diego (UCSD) with 18 specimens (six series of three specimens each: A, B, C, D, E, and F), and the Building and Fire Research Laboratory at the National Institute of Standards and Technology (NIST) with four specimens (two series of two specimens each: G and H). [Table 1](#) presents the characteristics of the test units in the following order: specimens label, cross-section dimensions (diameter of the circular columns D or side dimensions of the non-circular columns b and h), side aspect-ratio (h/b), overall specimens height (H), area of the gross section (A_g), area aspect-ratio ($A_g/A_{g(C)}$) (where $A_{g(C)}$ is the gross area of series C specimens), ratio of the area of longitudinal steel reinforcement to the cross-sectional area of the member (ρ_g), yield strength of longitudinal steel reinforcement (f_y), FRP volumetric ratio (ρ_f), and average concrete compressive strength at the corresponding age of testing of each column (f'_c).

Series A, B, C, D, E, and F consists of three specimens each: one control unit (A1, B1, C1, D1, E1, and F1), one unit strengthened to achieve an increment of 30% of load carrying capacity featuring a full wrapping scheme (A2, B2, C2, D2, E2, and F2), and a third unit (B3, C3, and D3) whose thickness of FRP jacket matched the same number of plies used in specimen A2. Specimens A3, E3, and F3 were partially wrapped for a 30% increment of carrying capacity as well. Series G and H were composed of two test units each: one control unit (G1 and H1), and one strengthened unit (G2 and H2) to gain 30% of increase in axial capacity. A gap of about 13 mm (0.5 in) was left at the top and bottom ends of the columns to avoid axial compressive loading of the FRP jacket. The partially wrapped specimens featured strips of 133 mm (5.25 in.) wide and a pitch of 76 mm (8.25 in.). Further details on the construction and strengthening of the specimens can be found in Rocca et al. (2006).

The detailing (concrete cover, longitudinal and transverse internal reinforcement layout, size, spacing, and shape of ties) of all the specimens was designed according to conventional RC practice (ACI 318-02). All the specimens featured a clear concrete cover of 38 mm (1.5 in.). The non-circular specimens were designed with a corner radius of 30 mm (1.2 in.). To prevent premature failure of the specimens at the top and bottom ends, steel transverse reinforcement spaced at about 50 mm (2 in.) was placed at these locations in all the specimens ([Figure 1](#)).

Material Properties

Concrete -- For the entire test program a nominal concrete compressive strength of 28 MPa (4,000 psi) representing the common strength in current building structures was considered appropriate. Since the specimens were cast at two different locations, the concrete constituents and properties are discussed separately.

All of the specimens at UCSD were built up from one single batch of ready-mix concrete having constituents and mix proportions as shown in [Table 2](#). Standard concrete cylinders 152×305 mm (6×12 in.) were prepared and cured under the same conditions of the columns. These cylinders were tested according to ASTM C39-04 at 7, 14, 21, 28 days, and at the corresponding age at which the related columns were tested (three cylinders per each case). The average compressive strength for the characteristic ages were 20.1 MPa (2.92 ksi), 23.7 MPa (3.44 ksi), 26.3 MPa (3.81 ksi), and 30.5 MPa (4.43 ksi), respectively.

The constituents and mix proportions of the concrete for the specimens at NIST are presented in [Table 3](#). Standard concrete cylinders were cast and tested at 7, 28 days and at the time of the actual testing of the columns. Due to the high congestion of the steel reinforcement at the top and bottom ends of the larger specimens (series G and H), a minimum slump of 20 cm (8 in.) was considered appropriate for the concrete to flow through the steel grids.

Reinforcing Steel -- Both UCSD and NIST specimens were designed with a Grade 60 (420 MPa) longitudinal steel reinforcement at a ratio of approximately 1.5%, [Table 1](#) shows the average yielding strength of the steel used in

each of the specimens. These average values were obtained from tensile tests on coupons performed according to ASTM A370.

CFRP -- Unidirectional CFRP of one-ply nominal thickness (t_f) of 0.167 mm (0.0067 in.) was the wrapping material used for the entire research project. For the preliminary design, the mechanical properties provided by the manufacturer were used. One and two-ply tensile coupon tests were performed to determine the mechanical properties of the CFRP material used in the evaluation of the test results (ASTM D3039-00). The characterization yielded an ultimate tensile strain ϵ_{fu} of 0.93%, an ultimate tensile strength f_{fu} of 2668 MPa (387 ksi), and a modulus of elasticity E_f of 291 GPa (42,200 ksi).

Instrumentation and Test Set-up

The instrumentation consisted of electrical strain gauges located on the longitudinal and transverse steel reinforcement, and on the FRP jacket at critical locations (corner areas and mid-distance on each side) along the perimeter of the cross-section on the central region of the strengthened specimens. Additionally, external sensors to measure axial deformation (linear potentiometers in UCSD specimens and LVDTs in NIST specimens) were fixed to the faces of the columns at about mid-height.

The equipment at both UCSD and NIST laboratories is capable of applying an axial compressive force of 53 MN (12,000 kip). However, because of the height limitation of the one at UCSD (1.5 m [5 ft]), the larger specimens (groups G and H) were tested at NIST. The loading was quasi-static and it was applied in cycles of increments of one fifth of the expected capacity of each specimen. The minimum load level during unloading corresponded to approximately 5% of the total expected capacity. See Figures 2 and 3 for illustrations on the test set-ups at UCSD and NIST laboratories, respectively.

TEST RESULTS

The test results of the columns are shown in Table 4. The table shows the maximum load for the unconfined case (P_{co}) or the increase in axial compressive loading (P_{cc}/P_{co}), the concrete compressive strength corresponding to the maximum load for the unconfined case (f'_{co}) or the strengthening ratio (f'_{cc}/f'_{co}), the compressive strength of concrete at ultimate (f_{cu}), the axial compressive strain at maximum load (ϵ'_c and ϵ'_{cc} for the case of unconfined and confined, respectively), the axial compressive strain at ultimate (ϵ_{cu} and ϵ_{ccu} for the case of unconfined and confined members, respectively), the ratio of ultimate axial strain of confined member to unconfined ($\epsilon_{ccu}/\epsilon_{cu}$), and the average transverse strain on the FRP jacket at ultimate (ϵ_{tu}). The axial strains are the average values obtained from linear potentiometers of LVDTs fixed to the sides of the columns. The transverse strain for the circular specimens corresponds to the average value of the measurements obtained by the sensors on the jacket. For the case of the non-circular specimens, it is the mean value of the readings given by the sensors closer to the corners.

The experimental values of f_{cu} and ϵ_{ccu} (or ϵ_{cu} for the unconfined case) were reported according to the following definition: ϵ_{cu} is the ultimate strain of the unconfined RC column corresponding to $0.85f'_c$ (See curve a - Figure 4). For the confined RC column, ϵ_{ccu} may correspond to one of the following values: a) $0.85f'_{cc}$ in the case of a lightly confined member (See curve b - Figure 4); b) the failure strain in the heavily confined, softening case when the failure stress is larger than $0.85f'_{cc}$ (See curve c - Figure 4); or the heavily confined, hardening case, where ultimate strength corresponds to ultimate strain (See curve d - Figure 4). The definition of ϵ_{ccu} (or ϵ_{cu} for the unconfined case) at 85% of f'_{cc} (or f'_c for the unconfined case) is arbitrary for cases corresponding to curves a and b, although consistent with modeling of conventional concrete (Hognestad 1951), and such that the descending branch of the stress-strain curve at that level of stress ($0.85f'_{cc}$ or higher) is not as sensitive to the test procedure in terms of rate of loading and stiffness of the equipment utilized (Rocca et al. 2007).

In Table 4, strain values from specimen H1 are not reported due to an inconvenience with the Data Acquisition System in the last load cycle applied to this specimen. Additionally, the values of transverse strain on the FRP jacket of specimens F2 and H2 seemed to be inaccurate, and therefore are not reported either.

The failure mode of the confined columns was characterized by rupture of the FRP jacket in the central region of the member. This was generally followed by buckling of the longitudinal steel reinforcement. In the case of the non-circular specimens, the FRP rupture took place at the corners of the section. See Figure 5 for illustrations.

For the specimens of non-circular cross-section the increment of axial compressive capacity (P_{cc}/P_{co}) is marginal compared to that of the circular counterparts. However, in Table 4, performance of specimens in series E appears to be an exception. This series featured the same characteristics as series F (material and geometrical) with the sole difference being the overall specimen height: 0.7 m (2.3 ft) and 1.4 m (4.6 ft), in the case of series E and series F, respectively. The observed higher ratios of P_{cc}/P_{co} of series E compared to the ones from series F may be due to the unexpected premature failure of control unit E1. The possible cause maybe the limited number of ties that was able to be placed along the height of the specimen. Additionally, stress concentrations induced at the top and bottom ends might have affected the overall strength as well. For these reasons, the experimental results of specimens in series E are believed to be not truly representative and therefore are not included in the analysis.

ANALYSIS AND DISCUSSION OF RESULTS

Figures 6(a), (b), and (c) present the performance of the specimens in terms of the ratio f'_{cc}/f'_{co} , which is a measurement of the confinement effectiveness, and the variables: side aspect ratio and area aspect ratio. In these figures, the label numbers within brackets represent the FRP volumetric ratio in percentages. Figure 6(a) shows the influence of the cross-sectional shape in the strengthening performance of specimens of a constant cross-sectional area ($A_g/A_{g(C)} = 1$). It can be observed that among specimens of different cross-sectional shape featuring similar FRP volumetric ratio (ρ_f), and taking the circular specimens with a ratio h/b equal to zero as a “benchmark”, the level of confinement effectiveness decreases as the side aspect ratio increases. Figure 6(b) and Figure 6(c) show the effect of the area aspect ratio for specimens of square and rectangular cross-sections, respectively. In these two figures, there seems to be decreasing confinement effectiveness trend as the area-aspect ratio increases, however, a definite conclusion cannot be drawn due to the limited amount of data available.

The results from this experimental program are also presented along with collected available data on RC columns of circular and non-circular cross-sections with one minimum dimension of the cross-section of 300 mm (12 in.), side aspect ratios not greater than 2, and FRP jackets with the fibers oriented perpendicular to the longitudinal axis of the column. The collected data on circular and non-circular RC specimens is shown in Table 5 and Table 6, respectively. Table 5 is composed of a total of 20 specimens divided in five sets of experiments, and Table 6 presents 13 specimens divided in six experimental sets. The specimens' codes correspond to the studies conducted by the following authors: “DN” to Demers and Neale (1994), “KE” to Kestner et al. (1997), “YO” to Youssef (2003), “CH” to Carey and Harries (2003), “MA” to Matthys et al. (2005), and “WR” to Wang and Restrepo (2001). Circular specimens “YO” were divided in two groups depending on the type of transverse steel reinforcement, that is: “-s” for spiral and “-h” for hoops. In Table 5 and Table 6, the data is presented in terms of the following parameters: type of FRP used, diameter of the circular cross-sections (D); side dimensions (b , h) and the chamfered corner radius of the non-circular specimens (r); overall height (H); longitudinal and transverse steel reinforcement ratio (ρ_g and ρ_t); FRP volumetric ratio (ρ_f); unconfined concrete compressive strength (f'_c); yield strength of the longitudinal and transverse steel reinforcement (f_y and f_{yt}); FRP mechanical properties (E_f , f_{fu} , and ϵ_{fu}); nominal ply thickness of FRP (t_f); maximum loads for the unconfined cases (P_{co}) or the increase in axial compressive loading (P_{cc}/P_{co}); and concrete compressive strengths corresponding to the maximum load for the unconfined cases (f'_{co}) or the strengthening ratio (f'_{cc}/f'_{co}).

All the experimental data is presented in Figure 7 in terms of trends of the strengthening ratio f'_{cc}/f'_{co} versus the parameter $\rho_f E_f / E_c$, which represents the ratio of the stiffness of the FRP jacket to the axial stiffness of the concrete. The product of the parameters ρ_f and E_f resembles the theoretical stiffness of the FRP jacket (E_j), also known as confinement modulus or lateral modulus (Xiao and Wu 2000; De Lorenzis and Tepfers 2001). This parameter represents the capacity of the FRP jacket of restraining the lateral dilation of the concrete. In the case of circular specimens, E_j is directly related to the maximum confining pressure as follows:

$$f_{l,f} = \left(\frac{2nt_f E_f}{D} \right) \epsilon_{fu} = \left(\frac{1}{2} \rho_f E_f \right) \epsilon_{fu} = (E_j) \epsilon_{fu} \quad (1)$$