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

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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 ºC (309 ºF), 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.

<u>As-built and retrofitted bridge piers – 1998 tests</u>

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 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 ¼ 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:

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

To maximize the contribution of the CFRP composite, the CFRP composite fabric fibers were placed at an angle of $\theta = (90^0 - \theta_p)$ from the longitudinal axis of the cap beam; in the in-situ application the sheets were laid at $\pm 45^0$ 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 45^0 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:

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

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:

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

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:

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

Two CFRP composite layers (L2) were provided on both sides of the cap beam at ± 45°, i.e. a total of four layers in the + 45° direction and four layers in the - 45° 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).

<u>Retrofitted bridge pier – 2000 tests</u>

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 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 ± 52° 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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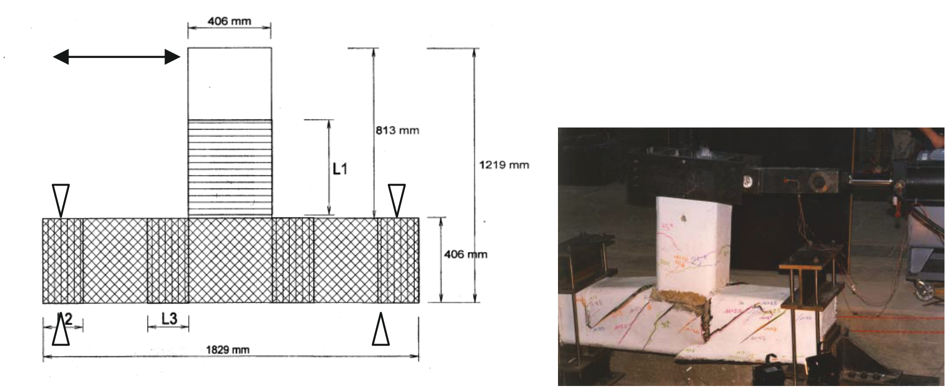

Figure 1 - CFRP composite layout for component beam-column joints (1 mm = 0.039 in.).

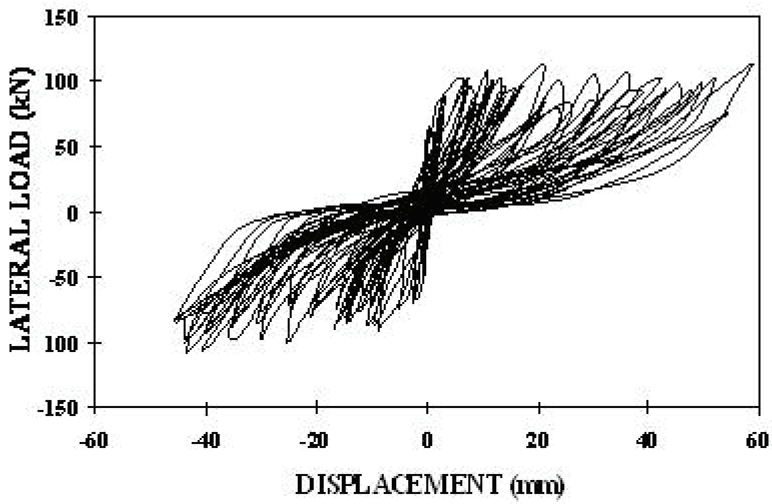

Figure 2 - Hysteresis for as-built beam-column joint.
(1 kN = 0.225 kips, 1 mm = 0.039 in.)

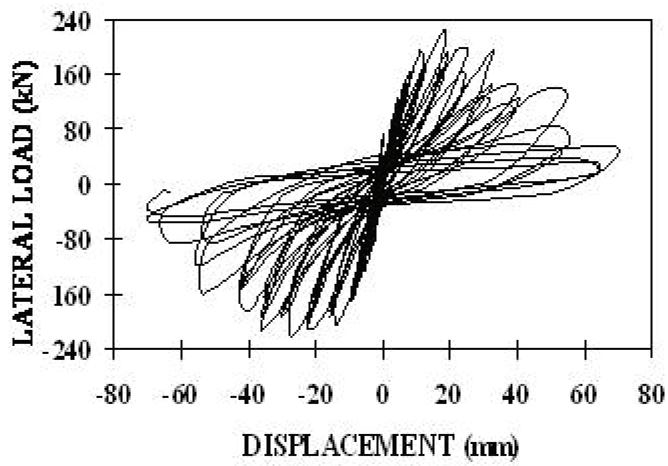

Figure 3 - Hysteresis for beam-column joint rehabilitated with CFRP composites.
(1 kN = 0.225 kips, 1 mm = 0.039 in.)

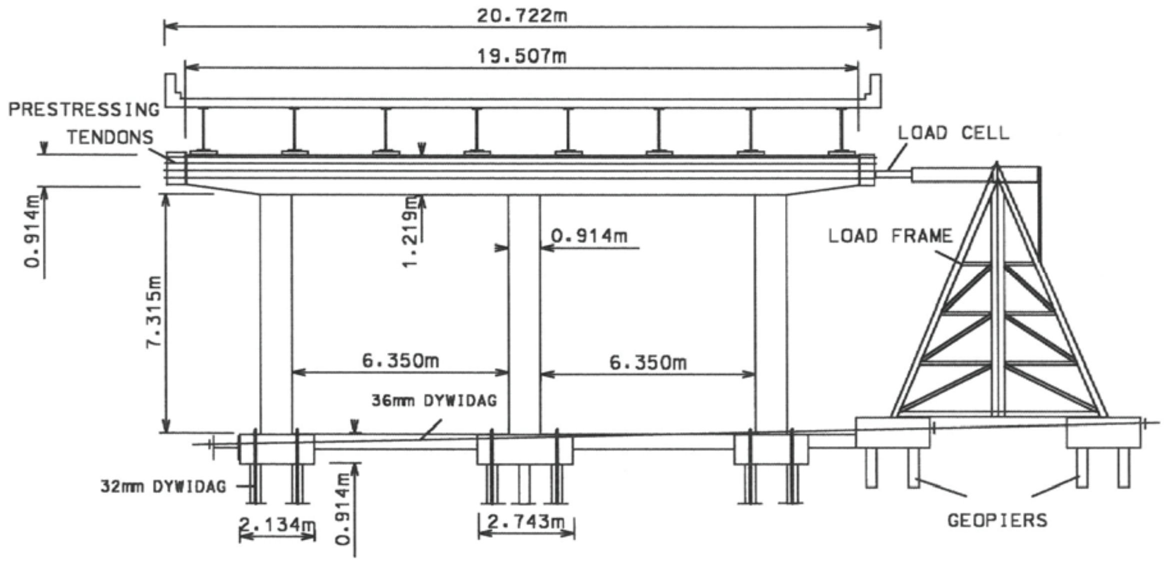

Figure 4 - Dimensions and loading condition for typical bridge pier - 1998 tests.
(1 m = 39.37 in., 1 mm = 0.039 in.)