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SEISMIC REHABILITATION OF REINFORCED CONCRETE BRIDGE COLUMNS IN MODERATE EARTHQUAKE REGIONS USING FRP COMPOSITES

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SUMMARY

Spectral design accelerations in the Northeast United States have significantly increased from previous values leading to higher seismic demands for structures constructed in this region. Under higher seismic demands bridge columns that were not designed to respond inelastically would now be expected to be able to accommodate inelastic displacements. More importantly, many of the bridges in the Northeast United States were designed before current earthquake design procedures were developed. Current seismic design practice of reinforced concrete structures promotes the use of adequate detailing to avoid premature failures of elements subjected to inelastic cyclic loading. Of particular concern are regions in structures where plastic hinges may form during the design seismic excitation.

The main goal of the experimental research presented in this paper was to develop and evaluate a method to rehabilitate the plastic hinge region of bridge columns for expected inelastic displacement demands characteristic of the Northeast United States. Fiber-reinforced polymer (FRP) composite materials have been used in the past to retrofit the plastic hinge region of bridge columns in California. Because bridge columns in the Northeast States are not expected to undergo the same level of inelastic seismic demands as required in the West Coast, the amount of FRP material used in the retrofit can be potentially reduced. The retrofitting schemes reported in this study consist of using two types of FRP systems, carbon and aramid-fiber laminates, wrapped in the plastic hinge region of ¼ scale models of typical bridge columns constructed in the early 1960s in the Northeast U.S. The thickness of the laminates was designed to achieve a ductility level representative of regions of moderate seismicity. The results indicate that economical retrofitting techniques using FRP jackets can be used to improve the seismic performance of bridge columns by increasing confinement and avoiding lap-splice failures near the base of existing columns.

INTRODUCTION

The use of FRP jackets for seismic retrofitting of deficient bridge columns is a technique that has been validated extensively through experimental testing. Significant improvements in the displacement

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ductility of columns with inadequate confinement have been reported [1, 2, 3]. Design procedures to determine the required jacket thickness for shear retrofit, column confinement, or lap-splice clamping were developed by Seible et al. [4]. The majority of these studies have focused on deficient bridge columns located in areas of high seismicity where the lateral displacement demands imposed by earthquakes require the use of thick FRP jackets or composite laminate fabrication procedures that promote active confinement of the column core [5]. The composite jackets in these reported studies were applied using specialized equipment or were prefabricated due to the required high curing temperatures. Although appropriate for structures located in high seismic regions due to the potential of large economic loss during a large earthquake, these techniques may not be cost-effective in regions of low seismic risk.

For regions of moderate seismic activity, the displacement demands imposed on bridge structures are much lower than in regions of high seismicity. The experimental studies presented in this paper were designed to evaluate the behavior of bridge columns rehabilitated using thin FRP jackets fabricated economically using a wet-layup procedure. The study focused on schemes designed to rehabilitate bridge columns through the local placement of FRP jackets in the potential plastic-hinge region to minimize the cost of retrofitting. In addition to providing confinement near the column base, the jackets were intended to avoid failure of lap splices typically located near the base of these older reinforced concrete bridge columns.

TESTING PROGRAM

Specimen Description

Six column specimens were fabricated and tested in the laboratory to investigate the effects of FRP wrapping in the behavior of bridge columns. The specimens were divided into two groups (I and II) of three columns depending on the axial load applied to the columns during testing. The axial load levels were selected as a lower and upper bound of loads expected in columns of a bridge prototype selected for this study. In each group one column was used as a control specimen and two were rehabilitated using FRP jackets of different materials applied near the base of the columns in the region of the potential formation of a plastic hinge.

The specimens were designed to replicate the response of columns typical of bridges constructed in the early 1960s in Massachusetts. The bridge prototype selected for this study is part of an overpass located in the outskirts of Boston and is representative of bridges built on the Eastern United States during this period. The reinforcing details conformed to those used in regions of low or moderate seismicity at that time. Details of the reinforcing details and rehabilitation configurations are given in the following sections.

Specimen Geometry and Reinforcing Layout

The specimens consisted of 240 mm (9.5 in.) diameter reinforced concrete columns with a 953 mm (37.5 in.) height tested in a cantilever configuration. The columns had bottom and top reinforced concrete blocks used to anchor the specimens to the reaction frame and to apply the lateral loading at the top of the columns, respectively. The longitudinal reinforcement consisted of 9 – 13 mm diameter bars (9- #4) distributed circumferentially around the column section, and a transverse reinforcing spiral made from 6-mm diameter (0.22 in.) deformed wire at a 76 mm (3 in.) pitch. The longitudinal reinforcement ratio, $\rho_l = A_s/A_g$, was 2.5% and the transverse volumetric steel ratio was approximately 0.008 for all the columns. The column longitudinal reinforcement was spliced at the base to the same number and diameter of reinforcing starter bars that were anchored into the foundation block. The starter bars extended 305 mm (12 in.) into the columns providing a splice length approximately equal to 24 d_b (Fig. 1). This splice length was representative of those found in bridge columns built in the late 1950s in Massachusetts. The average compressive strength of the concrete at the time of testing was 24 MPa (3400 psi), and the yield stress of the longitudinal and spiral reinforcement was 440 and 610 MPa (63.8 and 88 ksi), respectively.

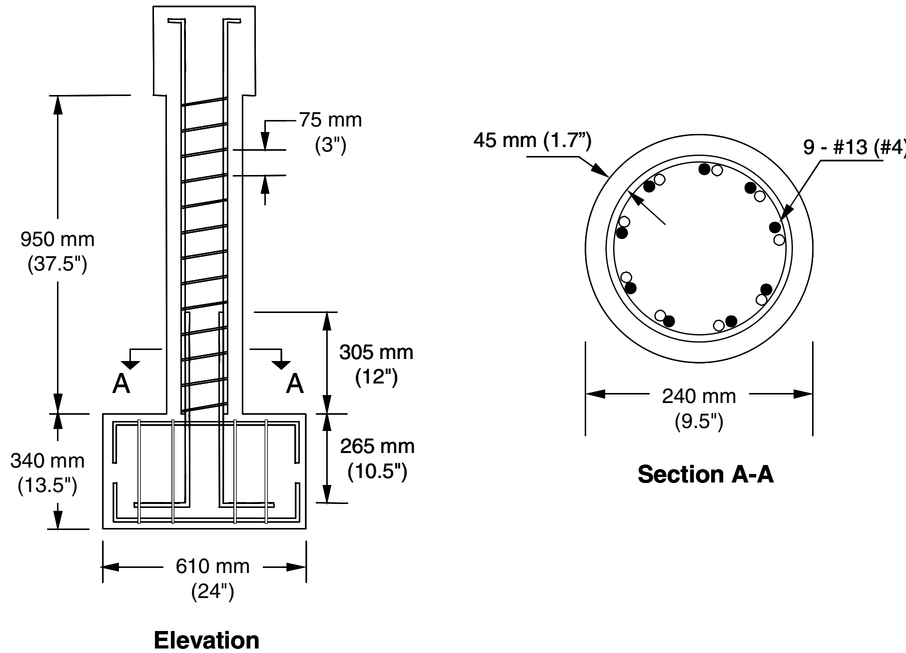


Figure 1 – Specimen Geometry and Reinforcing Details

Composite Strengthening Schemes

FRP wraps were used to provide concrete confinement and to eliminate the potential for lap-splice failure in the potential plastic hinge region of the columns. Significant lateral strength and stiffness degradation was expected to occur in the original columns at moderate displacement ductility levels because of the existence of a short lap splice at the base. The stiffness in the fiber direction of composite laminates plays a fundamental role in the confinement enhancement because of the linear stress-strain behavior of these materials. Therefore it was decided to investigate the confinement effectiveness of fiber composite jackets with different modulus and thickness. FRP composite jackets fabricated from carbon or aramid-fiber fabrics by wet layup were used to rehabilitate one of the specimens in each group of columns. The composites were formed using a single sheet of fabric in all the retrofitted specimens with an overlap of 150 mm (6 in.) in the hoop direction of the columns to avoid a localized failure at the fabric ends. The difference in composite stiffness was expected to affect the confining stress developed at a given lateral expansion of the concrete core. It was anticipated that the lateral confining stresses generated by the composites would preclude failure of the short lap-splice in the bottom region of the columns thereby increasing the deformation capacity and energy dissipation of the specimens. The fibers in the composite laminates were oriented in the hoop direction so that the contribution of the composites to the flexural strength of the columns was negligible. The jackets were started 6 mm (0.25 in.) above the foundation block and extended 345 mm (13.5 in.) above the base of the columns (Fig. 2). The jacket dimension was chosen to cover the entire lap-splice region of the columns. This rehabilitation scheme was selected to minimize cost of implementation for columns subjected to moderate displacement demands. Properties of the composite materials used in this study are listed in Table 1.

Specimens were designated using an alphanumeric label. Letters in this label correspond to either control specimens (C), specimens with carbon-fiber jackets (CFRP), or aramid-fiber (Kevlar) jackets (KFRP). The numbers following these characters correspond to the axial force applied during the tests, expressed as a percentage of the gross area of the columns times the nominal concrete strength of the specimens (05 or15).

Table 1 – Mechanical Properties of Composite Materials in the Fiber Direction (0º)

Composite Jacket Type	Design thickness, mm (in.)	Tensile strength, MPa (ksi)	Tensile rupture strain	Tensile modulus, MPa (ksi)
Carbon fabric	0.0165 (0.0065)	3,800 (550)	0.0167	227,000 (33,000)
Aramid fabric	0.0279 (0.011)	2,000 (290)	0.0155	120,000 (17,400)
Resin	—	55.2 (8)	0.035	3035 (440)

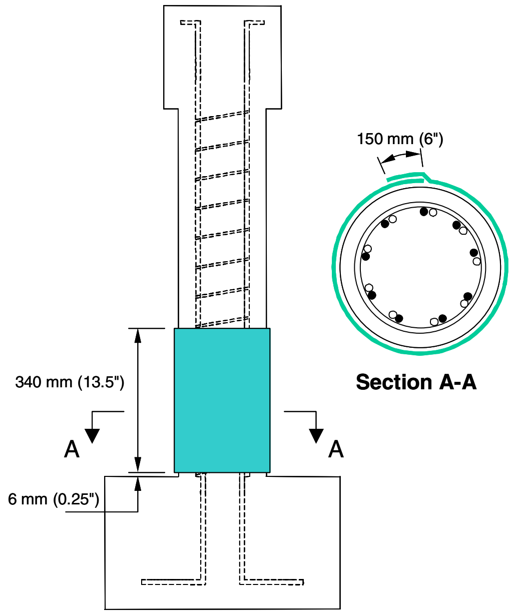


Figure 2 – Details of Column FRP-Jackets

Instrumentation

Three types of instruments were used in these tests: strain gages, linear potentiometers, and load cells. Strain gages were placed in the reinforcing steel, the concrete surface, and the FRP jacket surface. A dense array of these instruments was provided in the plastic hinge region of the columns. Strain gages were bonded to the longitudinal reinforcement in the columns to determine whether reinforcing bars reached yielding in the splice region during the tests. Likewise, strain gages in the transverse reinforcement were used to determine the confining force developed in the spiral reinforcement during the tests as a function of lateral displacement. Surface strain gages allowed determination of concrete or FRP jacket expansion at different ductility levels during the tests and also provided information on the confining efficiency of the FRP jackets.

Linear potentiometers were used to measure two different deformation parameters of the columns: (1) rotation in the plastic hinge region, and (2) lateral deflection along the column height. The rotation in the plastic hinge region near the base of the columns was determined using pairs of instruments located at four different heights on the north and south sides of the columns. The south side of the specimens corresponded to the side where the actuator was mounted. These instruments were fixed to threaded rods that extended through the column core to avoid disruption of readings after concrete cover spalling. Five potentiometers were distributed along the height on the north face to determine the deformed shape of the columns during testing.

Two load cells were used in the tests. A 98 kN (22 kip) load cell was positioned between the actuator and the top block of the specimens to measure the applied lateral load. An 89 kN (20 kip) load washer was used to measure the axial load applied to the columns by measuring the tensile force in high strength rods used to apply the axial force to the specimens.

Test Setup and Loading Protocol

The experimental setup was designed to simulate single curvature bending representative of flexure of columns in the longitudinal direction of the bridge. Specimens were constructed on a stiff base block to simulate rotational fixity of the columns. A top block was provided to transfer the axial and lateral loads to the specimens. Lateral loads were applied using a 98 kN (22 kip) MTS servo-controlled actuator centered on the top block of the specimens located at a distance equal to 1085 mm (42.75 in.) from the foundation block (Fig. 3).

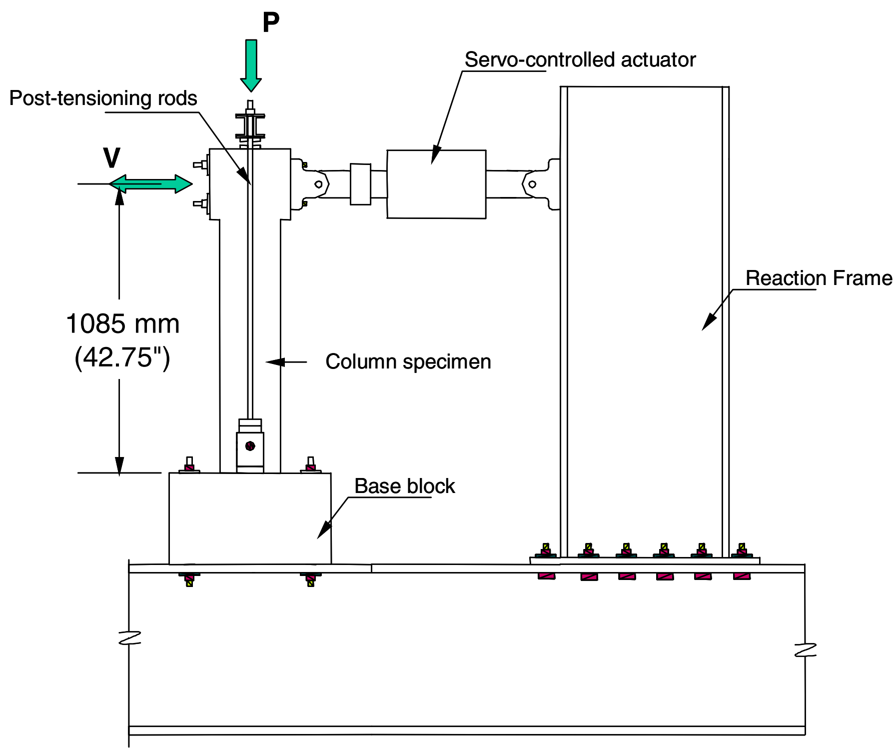


Figure 3 – Experimental Setup

Axial load was applied at the start of the tests and maintained constant throughout the lateral loading protocol. The axial load was applied to the top block of the columns by post-tensioning steel rods located on both sides of the specimens. These rods were anchored to a steel assembly resting on the top block and to a pinned swivel plate anchored to the foundation block. The post-tensioning assembly did not restrain the columns from translating laterally. The axial load applied to each column group was either 5% or 15% of the gross area (A_g) times the nominal compressive strength of the concrete (f'_c).

Cyclic lateral loading amplitude was increased sequentially during the tests. Load controlled cycles of 0.5 and 0.75 times the yield lateral load (V_y) were first applied, followed by displacement controlled cycles of amplitude equal to the yield displacement (Δ_y), 1.5 Δ_y, 2 Δ_y, 3 Δ_y, etc. until failure of the columns was reached. The yield load of the specimens was calculated using the measured steel and concrete properties of the control specimens. For all specimens within a group, the yield displacement was assumed equal to the measured displacement of the control specimen at the calculated yield load.

Displacement ductility was therefore defined as the measured displacement divided by the yield displacement of the control specimens within each group. The load or displacement amplitude was increased to the next level after applying three full cycles of loading at a given level. Loading was stopped when either the stroke of the actuator was reached or when failure of the specimens occurred as evidenced by significant loss of lateral strength. For this study, lateral-load failure of the columns was defined when the load during the first cycle at a given displacement level dropped below 80% of the peak load measured during the previous loading amplitude.

TEST RESULTS

In this section the behavior of specimens within each column group is discussed to evaluate the effects of using different composite materials in the column rehabilitation. The effect of axial load level is presented by comparing the response of specimens in different groups rehabilitated using identical composite materials. Both global and local response parameters are discussed and compared. For comparison of results, the lateral load was normalized with reference to the yield load of the control specimens within each group, while the measured displacement response is presented in terms of the overall column drift (displacement at the actuator level divided by the distance to the base).

Load-Deflection Response

Failure of the control specimens for both column groups was triggered by a lap-splice failure at the base of the columns at relatively low lateral displacement ductility. Longitudinal cracks started forming between flexural cracks on the concrete surface during the load cycles corresponding to yielding of the columns (Fig. 4a). Lateral load-drift response of the control columns for Groups I and II are shown in Fig. 5a and 6a, respectively. It can be observed that failure occurred at drifts between 0.05 and 0.075 in these columns. Significant strength degradation was observed during the second and third cycles at 2 Δ_y and failure occurred at 3 Δ_y. Lateral strength loss was observed at 2 Δ_y for specimen C-15 with only about 8% of the yield load remaining at a displacement ductility of 3. The lateral-load drift response of these columns was characterized by significant pinching during the cycles at 2 and 3 Δ_y indicating the limited energy absorbing capacity of these columns.

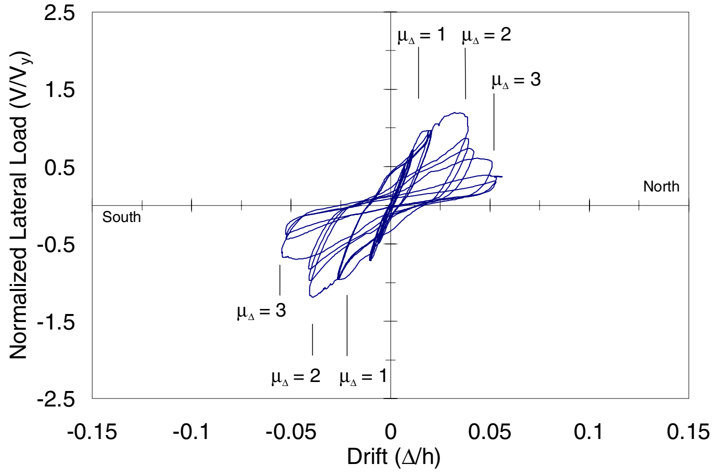
(a) Specimen C-15 (b) Specimen KFRP-15

Figure 4 – Observed Failure Modes of Original and Retrofitted Specimens

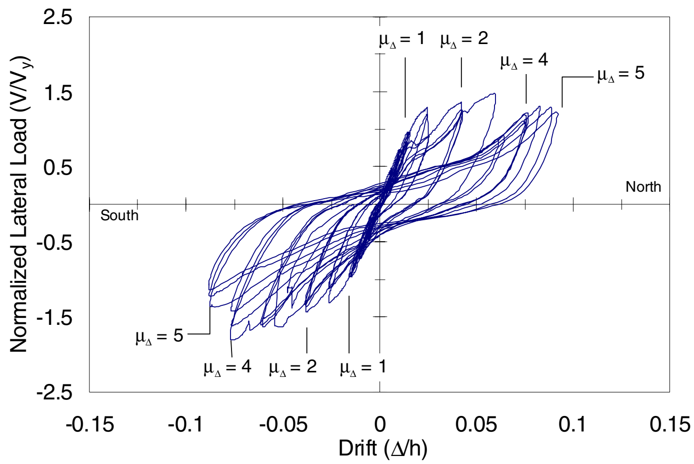
The columns rehabilitated with either carbon or aramid composite jackets maintained their lateral strength to significantly higher drifts than the corresponding control specimens (Figs. 5 and 6). This behavior was a result of a change in the observed failure mode from a non-ductile splice failure to a ductile flexural failure of the rehabilitated columns. The behavior of these columns was controlled by the formation of a plastic hinge within the lap-splice region (Fig. 4b) or directly above the composite jackets. The plastic hinge formed in the FRP-wrapped region of the column in three of the four rehabilitated specimens (specimens CFRP-05, KFRP-05, and KFRP-15) as determined from the recorded strain data. These specimens maintained their lateral-load strength approximately to a displacement ductility of 5. The plastic hinge in specimen CFRP-15, however, formed above the FRP jacket (Fig. 7), which limited the displacement ductility of this specimen to 4. In contrast to the control specimens, the lateral strength of all of the rehabilitated specimens increased significantly after yielding as a result of the confining effect provided by the FRP jackets. Higher strength increases were observed for the rehabilitated columns in Group I (approximately 40%) than for columns in Group II (between 19 and 30%). It is important to account for this increase in lateral strength when evaluating other members connected to the rehabilitated element in the structure. Table 2 lists the main load-drift parameters of all the specimens tested in this project.

Table 2 – Lateral-load Drift Parameters

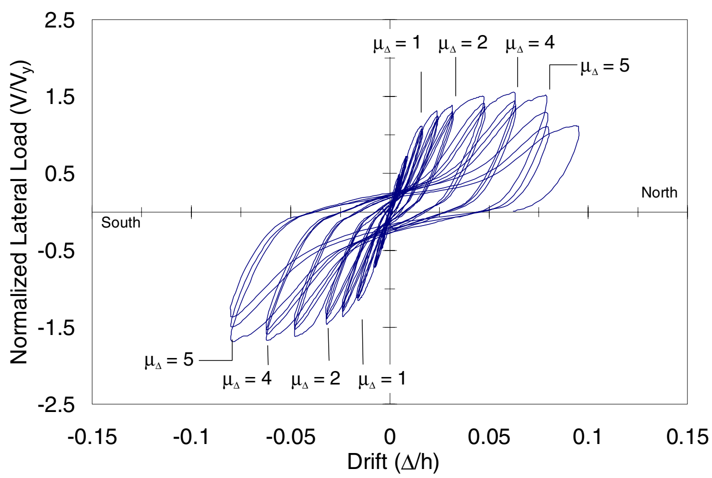
Specimen	V_y, kN (kip)	Drift at yield (Δ_y/h)	V_{max}, kN (kip)	V_{max}/V_y	Drift at peak load	Maximum drift (Δ_{max}/h)	Displacement ductility (μ_Δ)
Group I – $P = 0.05\ A_g\ f'_c$							
C-05	32.9 (7.4)	0.017	39.6 (8.9)	1.21	0.035	0.051	2
CFRP-05	32.9 (7.4)	0.016	46.3 (10.4)	1.41	0.060	0.092	4
KFRP-05	36.5 (8.2)	0.016	50.3 (11.3)	1.38	0.063	0.095	5
Group II – $P = 0.15\ A_g\ f'_c$							
C-15	36.5 (8.2)	0.025	36.5 (8.2)	1.00	0.039	0.050	2
CFRP-15	46.3 (10.4)	0.024	55.2 (12.4)	1.19	0.060	0.092	4
KFRP-15	40.0 (9.0)	0.023	52.0 (11.7)	1.30	0.062	0.100	5



(a) C-05

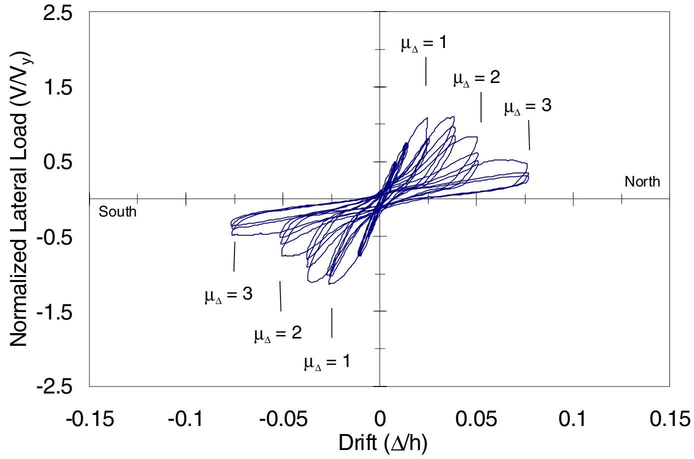


(b) CFRP-05

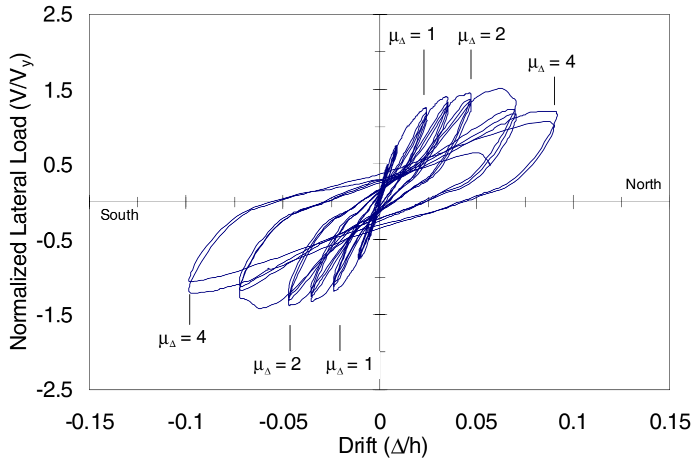


(c) KFRP-05

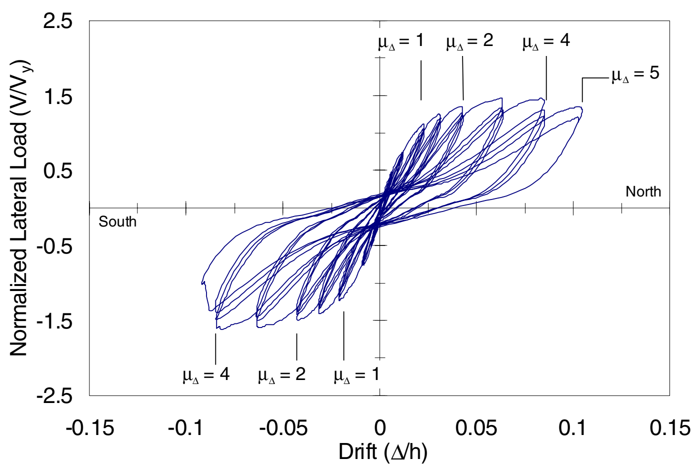
Figure 5 – Lateral-Load Drift Response of Specimens in Group I ($P/A_g f'_c = 0.05$)



(a) C-15



(b) CFRP-15



(c) KFRP-15

Figure 6 – Lateral-Load Drift Response of Specimens in Group II ($P/A_g f'_c = 0.15$)

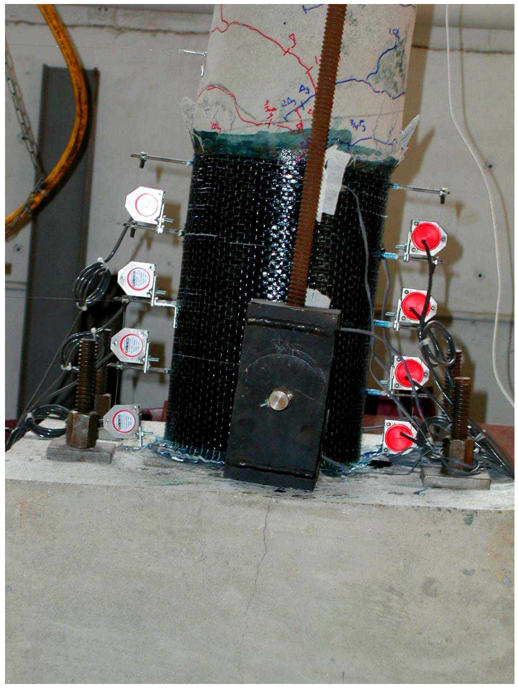


Figure 7 – Formation of Plastic Hinge Above Jacket in Specimen CFRP-15

Strains

The specimens were instrumented with strain gages at various locations near the base of the columns. Only the strains measured in the transverse reinforcement and the FRP composite jackets are discussed in this section because of their relevance in concrete confinement. Strain gages were bonded to the transverse reinforcement on the east and west sides of the columns at three different heights along the lap-splice near the base of the columns. Similarly, strain gages were positioned on the surface of the FRP jackets on the east and west sides of the columns at three heights corresponding to the locations of the transverse steel gages (Fig. 8).

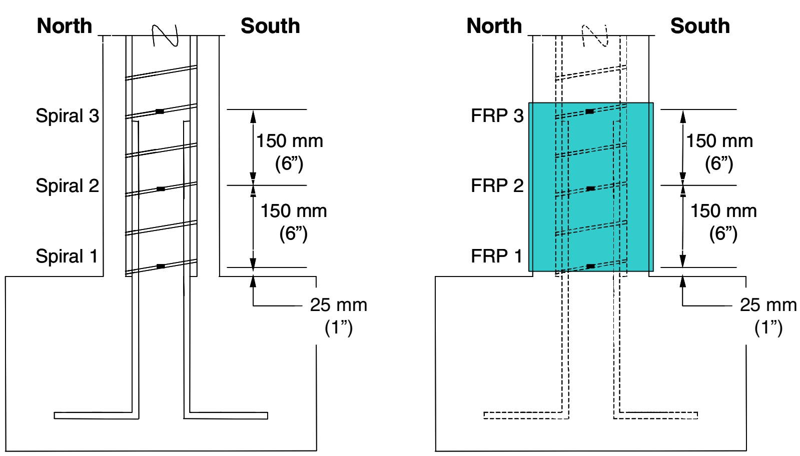
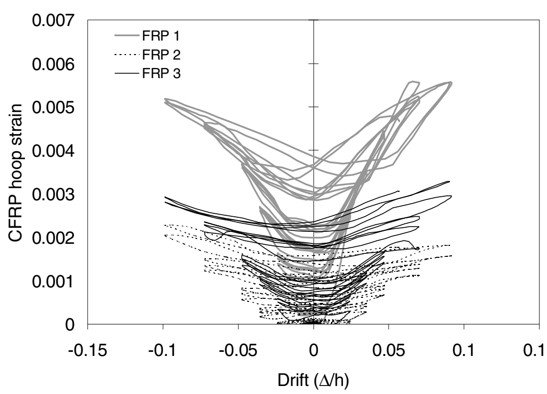
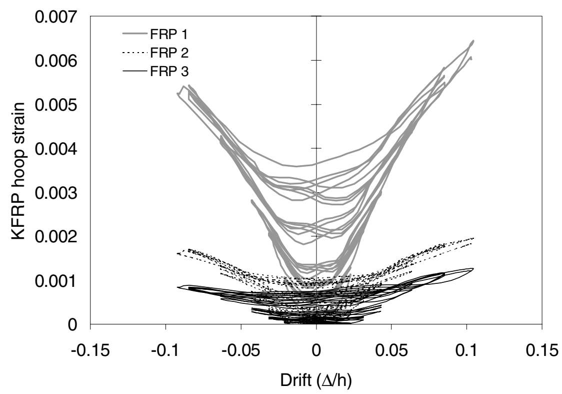


Figure 8 – Strain Gage Location in Transverse Reinforcement and FRP Jackets

Figure 9 illustrates the variation of strain on the surface of the composite jackets in the hoop direction as a function of column drift for the rehabilitated specimens in Group II. Results from the specimens in Group I were similar so are therefore not presented in this paper. Each figure shows three plots corresponding to the average of two strain gages at each of the three locations shown in Fig. 8. The instruments located closest to the base of both specimens (FRP 1) registered the highest strain as expected. The maximum hoop strain developed in the composite jackets near the base of the columns ranged from 0.0055 to 0.0065 at a drift of approximately 0.1 in both loading directions. The FRP hoop strains were expected to be highest at the base and decrease toward the top of the jackets as a result of lower moment in the columns. It should be recalled that these strains generate as a result of expansion of the concrete when subjected to axial strains. The FRP hoop strains in specimen KFRP-15 followed the expected trend in the results, with the measured strain at the top of the FRP jacket reaching values of 0.001 and 0.0015 at the maximum drift of the column. On the other hand the FRP hoop strains measured in specimen CFRP-15 near the top of the FRP jacket (FRP 3) were higher than those measured near the middle of the jacket (FRP 2). This result can be attributed to the formation of a plastic hinge above the composite jacket in specimen CFRP-15.



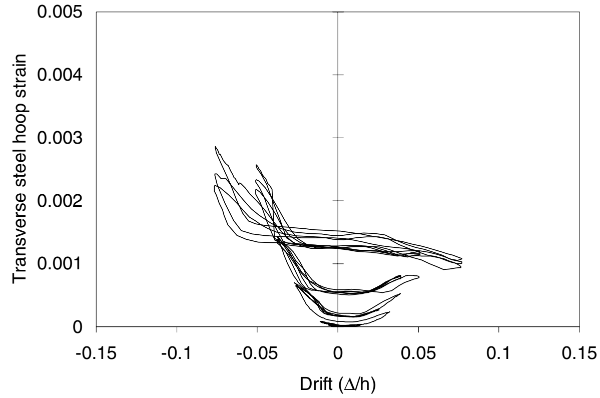
(a) Specimen CFRP-15



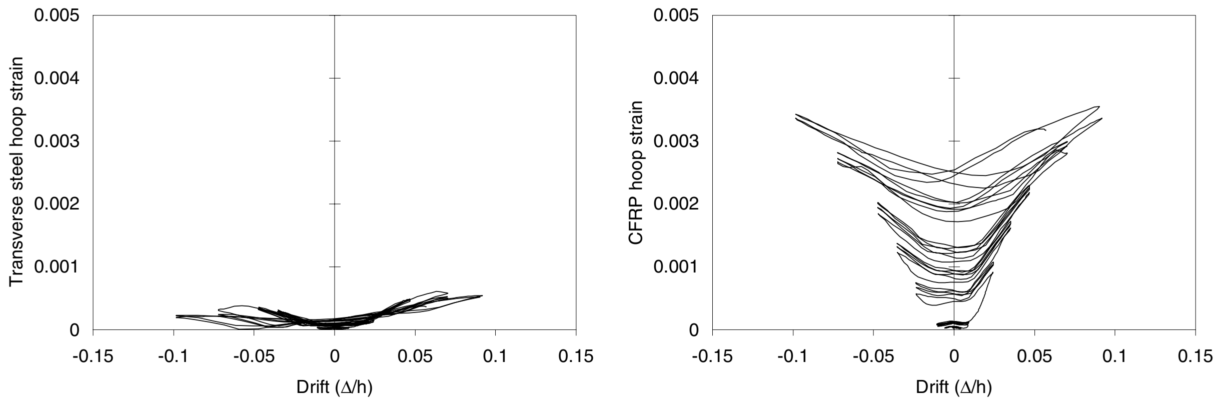
(b) Specimen KFRP-15

Figure 9 – Variation of FRP Hoop Strains along Height of Jacket

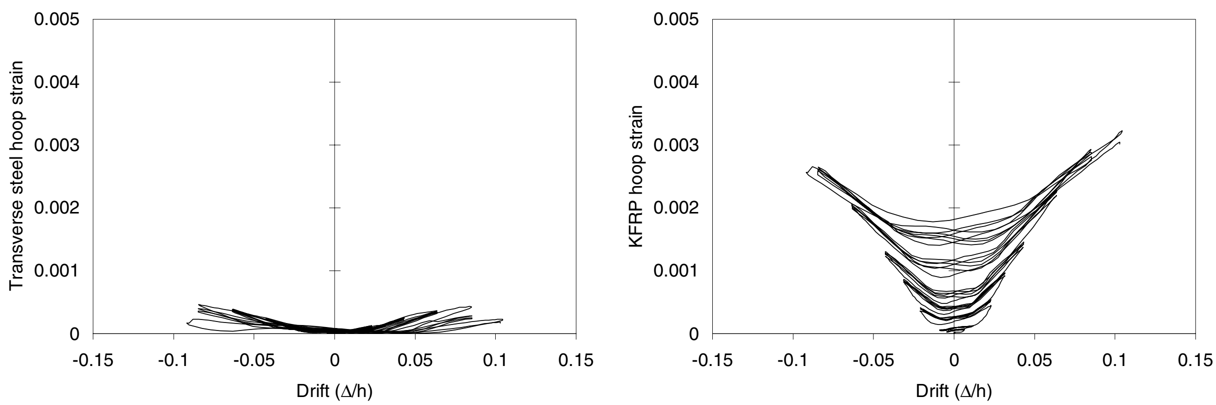
The efficiency of the composite jackets was assessed by comparing the strains developed in the spiral reinforcement of the control specimens with the strains developed in the composite jackets in the rehabilitated specimens. A comparison of the hoop strains developed in the reinforcing spiral and the composite jackets of specimens in Group II is presented in Fig. 10. The variation of strains along the height of the jacket presented in Fig. 9 was similarly observed in the spiral reinforcement strains. For this reason the plots shown in Fig. 10 were constructed using the average strains recorded at the three instrument locations shown in Fig. 8. These plots represent the average strains measured over the entire wrapped region of the columns. The spiral steel strains measured in specimen C-15 are shown in Fig. 10a. The peak spiral strains when loading in the north and south directions of the column were approximately 0.001 and 0.003, respectively, at a drift of approximately 0.075. A comparison between Fig. 10a and Figs. 10b and 10c reveals that a significant decrease in the spiral steel strains was observed in the rehabilitated specimens as a result of the confinement provided by the FRP jackets. Spiral strains in these columns reached values of only 0.0003 to 0.0007 at column drifts of up to 0.1. Interestingly, the spiral strain curves in the rehabilitated specimens exhibit shallower slopes than the curve in specimen C-15 indicating less expansion of the concrete core at large drifts. The contribution of the FRP jackets to concrete confinement is clear when observing Figs. 10b and 10c. The average FRP hoop strains in the lap-splice region of the specimens reached values of between 0.0028 and 0.0035 at a column drift of approximately 0.1. Table 3 lists the average hoop strains measured in the spiral and FRP jackets for loading in the north and south directions at the maximum drifts attained in the specimens before failure.



(a) Specimen C-15



(b) Specimen CFRP-15



(c) Specimen KFRP-15

Figure 10 – Comparison of Average Hoop Strains in Spiral and FRP Jackets

The plots presented in Fig. 10 can also be used to evaluate the development of confining stresses from different available mechanisms (steel spiral or FRP jacket) as a function of column drift. Confining stresses generated by the steel spiral, f_{ls}, or the FRP jackets, f_{lj}, were calculated using the models shown in Fig.11, where E_s and E_j are the elastic moduli of the steel and FRP jackets, respectively; ε_s and ε_j are the average measured hoop strains in the steel and FRP; A_v is the area of the reinforcing spiral at a pitch s;

and the remaining parameters are defined in the figure. These confining stresses give an indication of the contribution of the available mechanisms to the generation of the total confining stresses in the columns. The confining stress values listed in Table 3 clearly indicate a reduction in the contribution of the spiral steel to the total confining stress for the rehabilitated specimens. Because the FRP jackets are applied externally, the entire cross section is confined effectively and the possibility of reinforcing bar buckling in minimized.

Table 3 – Hoop Strains Measured in Wrapped Region of Columns

Specimen	Average strains in confined column region		Average confining stress, MPa (psi)		Contribution to confining stress, %	
	Spiral	FRP jacket	Spiral	FRP jacket	Spiral	FRP jacket
C-05	0.0009	—	0.8 (112)	—	100	—
CFRP-05	0.0004	0.0023	0.4 (52)	0.7 (104)	33	67
KFRP-05	0.001	0.0027	0.9 (129)	0.8 (109)	54	46
C-15	0.0014	—	1.2 (180)	—	100	—
CFRP-15	0.0006	0.0034	0.5 (74)	1.1 (154)	32	68
KFRP-15	0.0004	0.0028	0.4 (52)	0.8 (113)	32	68

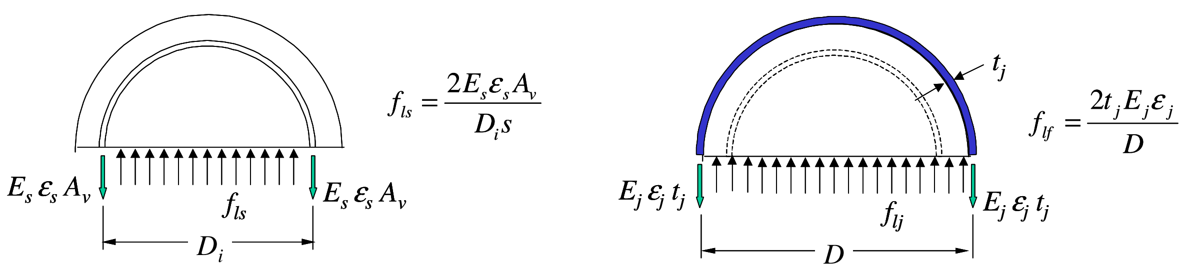
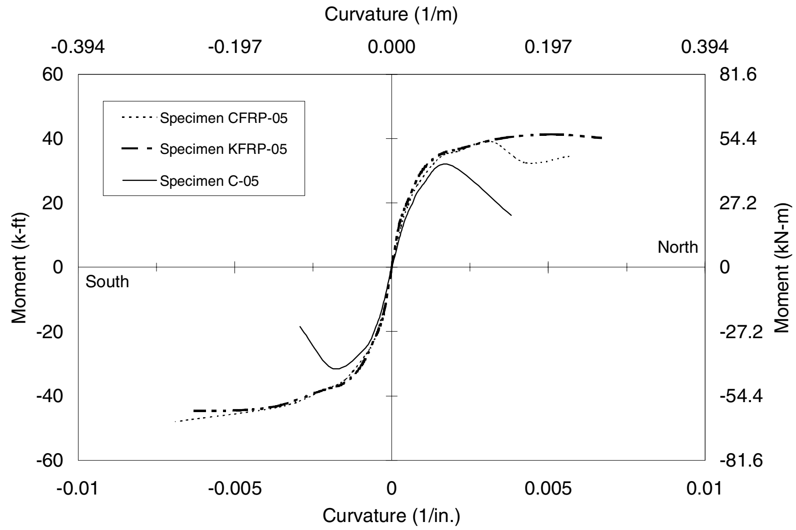


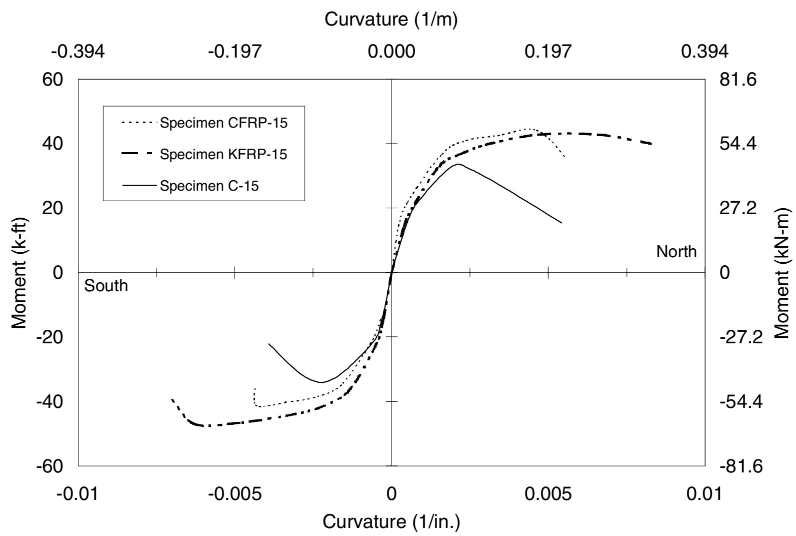
Figure 11 – Confining Stress Contribution of Spiral and FRP Jacket

Moment-Curvature Response

The section ductility of the specimens was evaluated by comparing the moment-curvature response of the specimens within the plastic hinge region. The rotation of the columns was measured at four sections near the base of the columns. The average curvature between two sections was calculated by dividing the rotation difference at two consecutive sections by the distance between the sections. The average curvature over the plastic hinge region was calculated as the average of the curvatures determined for each section. The values of moment and average curvature determined during the first cycle at each displacement level applied during the tests were used to develop the moment-curvature envelopes shown in Fig. 12. The curves shown in Fig. 12a correspond to the columns subjected to 5% $A_g\, f'_c$ while the curves shown in Fig. 12b correspond to columns subjected to 15% $A_g\, f'_c$. These figures show that the composite jackets had a significant influence in the increase of the curvature capacity of both groups of columns. The curvature capacity of the jacketed specimens increased 3 to 4 times from the curvature at the peak measured moment of the control specimens. The plots also show that in general specimens rehabilitated using aramid-fiber jackets developed slightly higher curvature over the plastic hinge length than specimens with carbon-fiber jackets.



(a) Group I



(b) Group II

Figure 12 – Comparison Between Moment-curvature Response of Control and Rehabilitated Specimens

CONCLUSIONS

Experimental results of ¼-scale bridge columns rehabilitated using FRP jackets for regions of moderate seismicity were presented. The columns tested in this research have details representative of common design practice of the early 1960s in Massachusetts. Two primary detailing deficiencies associated with poor seismic performance were found in these elements: insufficient transverse reinforcement for confinement and lapped reinforcing bars near the base of the columns. This study focused on rehabilitating the region near the base of the columns using FRP composite jackets where inelastic action is expected to occur.

The results of this study show that FRP jackets fabricated using a wet-layup procedure can be used to rehabilitate columns and change the failure mode from a non-ductile lap-splice failure at the base to a ductile plastic hinge failure mode. Increases in displacement ductility from 2 to 5 were observed in the rehabilitated columns for the two axial load levels tested in this research. The displacement demands in the regions where these columns are built are expected to be lower than these values. Additionally the FRP jackets provided sufficient confinement within the plastic hinge region of the columns to increase the lateral strength between 19 and 40%. A significant reduction in the lateral confining stress provided by the reinforcing steel spiral was also observed in the rehabilitated columns. The integrity of the rehabilitated columns was preserved by the FRP jackets thereby controlling the possibility of longitudinal reinforcing bar buckling at large displacements.

REFERENCES

1. Saadatmanesh H, Ehsani MR, and Jin L. “Seismic strengthening of circular bridge pier models with fiber composites.” ACI Structural Journal 1996; 93 (6): 639-647.
2. Saadatmanesh H, Ehsani MR, and Jin L. “Repair of earthquake-damaged RC columns with FRP wraps.” ACI Structural Journal 1997; 94 (2): 206-215.
3. Xiao Y, Ma R. “Seismic retrofit of RC circular columns using prefabricated composite jacketing.” Journal of Structural Engineering 1997; 123 (10): 1357-1364.
4. Seible F, Priestley MJN, Hegemier GA, and Innamorato D. “Seismic retrofit of RC columns with continuous carbon fiber jackets.” Journal of Composites for Construction 1997; 1(2): 52-62.
5. Seible F, Priestley MJN, and Innamorato D. “Earthquake retrofit of bridge columns with continuous carbon fiber jackets.” Advanced Composites Technology Transfer Consortium, Report no. ACTT-95/08 1995; 3.13-3.39.

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