Seismic Retrofitting of Rectangular Bridge Columns for Shear

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Retrofitting measures applied to ⅔-scale shear-deficient columns representative of existing rectangular bridge columns in the Puget Sound area of Washington state were investigated. The retrofit methods studied included external hoops applied over the height of the column and full-height rectangular steel jacketing. Test specimens consisted of a single column connected at the base to a rectangular footing. The specimens were subjected to increasing levels of cycled inelastic displacements under constant axial load. Performance of the specimens was evaluated in terms of load capacity and ductility. Tests on the column representing as-built conditions resulted in a brittle shear failure at the calculated yield displacement, that is, at a displacement ductility level of $\mu = 1$. Both retrofit methods investigated improved the behavior of the deficient column. With the external hoop retrofit, performance of the retrofitted columns was only moderately improved over that of the as-built column. Brittle fracture of the retrofit hoops limited the load-carrying capability and ductility enhancement, with displacement ductility levels of $\mu = 2$ and 4 being achieved. With the use of the rectangular steel jacket retrofit, performance was significantly improved over that of the as-built column. The jacket retrofit resulted in a ductile column response with good load-carrying capability through $\mu = 8$. Application of this retrofit over the full height of the column enabled the steel jacket to increase the column shear strength so that a flexural failure mode resulted. Although buckling of the steel jacket and longitudinal reinforcement occurred near the maximum moment section, sufficient confinement to the hinging region was provided by the buckled steel jacket to maintain load-carrying capability.

The extensive damage to bridge structures in the 1971 San Fernando earthquake caused a significant reevaluation of the seismic design approach for bridges. Since then, improvements have been incorporated into current design criteria. However, many bridges were built before the introduction of these new standards. Bridge failures in California and Alaska under relatively moderate earthquake loadings and, most notably, the collapse of the I-880 freeway in the Loma Prieta earthquake clearly indicate the vulnerability of older bridges and the need to develop methods for strengthening these bridges to meet current safety requirements.

In the United States, much of the work on seismic retrofitting of bridges has been done in California. Significant retrofit efforts began there in the late 1970s, with the focus of the retrofit program being to improve the performance of superstructures in an earthquake. Only relatively recently did the California Department of Transportation (Caltrans) begin retrofitting bridge substructures. It is notable that many of the bridges that experienced substructure damage during the Loma Prieta earthquake had movement restrainers installed in the superstructures. Clearly, retrofit efforts must address the entire structure before adequate structural safety can be achieved.

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A common problem in pre-1971 bridges is an insufficient amount of transverse reinforcement in the columns. Typically, No. 3 or No. 4 transverse hoops spaced at 12 in. on center were used in rectangular columns, regardless of column cross-section dimensions, and the hoops had short hook extensions and anchorage only by lapping the ends in the cover concrete. Further, intermediate ties were rarely used. This detail results in many older columns being susceptible to shear failures, and it provides little confinement for developing full flexural capacity or preventing buckling of the longitudinal reinforcement.

The objective of the research presented in this paper was to identify retrofit techniques for increasing the shear strength and ductility capacity of rectangular reinforced concrete bridge columns. A detailed account of the research program can be found elsewhere (*1*). This paper presents an overview of the study and discussion of the test results and conclusions.

BRIDGE COLUMN RETROFITTING

Previous Retrofit Research

Chai et al. (*2,3*) examined the effectiveness of retrofitting circular and rectangular bridge columns with circular and elliptical steel jackets, respectively, in which the gap between the jacket and column was filled with high-strength grout. Initially, the jacket was used only in the plastic hinge region and terminated just above the footing. As-built circular columns with lapped starter bars did not reach their theoretical strength because of bond failure in the early stages, after which the stiffness and strength degraded quickly. A comparable column retrofitted with a 3/16-in.-thick circular steel jacket showed tremendously improved results. In tests of as-built rectangular columns with lapped starter bars, there was bond failure at the splice leading to rapid strength and stiffness degradation. When retrofitted with a 3/16-in.-thick elliptical jacket, excellent hysteretic response was achieved. A later phase of testing showed that the same circular and elliptical steel jackets, extended over the full height of the columns, were effective in enhancing the shear strength and ductility in circular and rectangular columns, respectively.

Coffman et al. (*4*) studied a retrofit method for improving bridge column ductility that used external hoops, prestressed with turnbuckles, around the lower portion of circular columns. This scheme resulted in a dramatic increase in the total energy dissipation of the section and an increase in seismic durability by an order of magnitude over the as-built column. This method appears to improve the force transfer between the dowels and longitudinal steel in the splice region, even under repeated inelastic displacements.

Bett et al. (*5,6*) improved the performance of rectangular columns by adding external longitudinal reinforcement and closely spaced ties. This retrofit method was varied by adding cross ties through the column that were anchored by hooking around the longitudinal reinforcement. The cross ties improved the confinement, resulting in decreased strength and stiffness degradation under reversed cyclic loading.

Fyfe and Priestley (*7*) studied a retrofit method that used a high-strength fiber-reinforced fabric that was posttensioned around the plastic hinging region of a column. This retrofit method enhanced the flexural ductility and prevented the bond failures that were observed in the as-built columns tested by Priestly et al. (*2,3*).

Current Retrofit Practice

Currently, very little information exists on standard procedures for retrofitting bridge substructures. In the United States, only Caltrans has implemented any standardized procedures for selection of critical substructure elements and specifications for retrofitting once a bridge substructure has been identified as critical (*8*).

In bridges with columns identified as being unsatisfactory, Caltrans has standardized two column retrofit methods, the Class P retrofit and the Class F retrofit, which consist of ⅜-in.-thick steel jackets around the columns. For shear-deficient columns, a full-height jacket is used. Circular or elliptical jackets are used, depending on whether the column is circular or rectangular. The Class P retrofit provides partial confinement in the plastic hinging regions and only modifies the column, whereas the Class F retrofit modifies both the column and the footing, resulting in higher costs with the Class F retrofit. For this reason, a common starting point in Caltrans retrofit strategy is to use a Class F retrofit on one column per frame and Class P retrofits on the other columns in the frame.

EXPERIMENTAL TESTING PROGRAM

A representative bridge column from the Puget Sound area of Washington state was identified and used as the deficient specimen to which retrofit measures were applied and evaluated. The prototype column was formulated by compiling design information from existing Washington state bridges and identifying common dimensions, reinforcement arrangements, and deficient details in the columns. The column chosen was a 20- × 30-in. section with reinforcement concentrated on the 20-in. faces and with a total reinforcing ratio of 2.6 percent. The column contained No. 3 transverse hoops at 12 in. on center with 4-in. hook extensions that were lapped in the cover concrete for anchorage. All reinforcement in the column was Grade 40, which was used almost exclusively in the older bridges being studied.

The experimental tests were conducted on ⅖-scale specimens that modeled the dimensions, reinforcement content and arrangement, deficient detailing, and material properties of the chosen prototype column. A cross section of the scaled specimen is shown in Figure 1, which represents the control specimen to which all retrofitted column results were referenced. The test specimens consisted of a single column connected at the base to a rectangular footing. To prevent a footing failure that would introduce another variable into the testing program, the footing was designed to be stronger than those commonly found in pre-1971 designs. Continuous longitudinal bars extended into the footing using 90-degree hooks for resistance to pullout (see Figure 1). Tests were performed on parallel sets of specimens: one specimen incorporated deficient as-built detailing (the control specimen), and the other incorporated the same detailing but was retrofitted so that the benefit of the retrofit could be clearly seen.

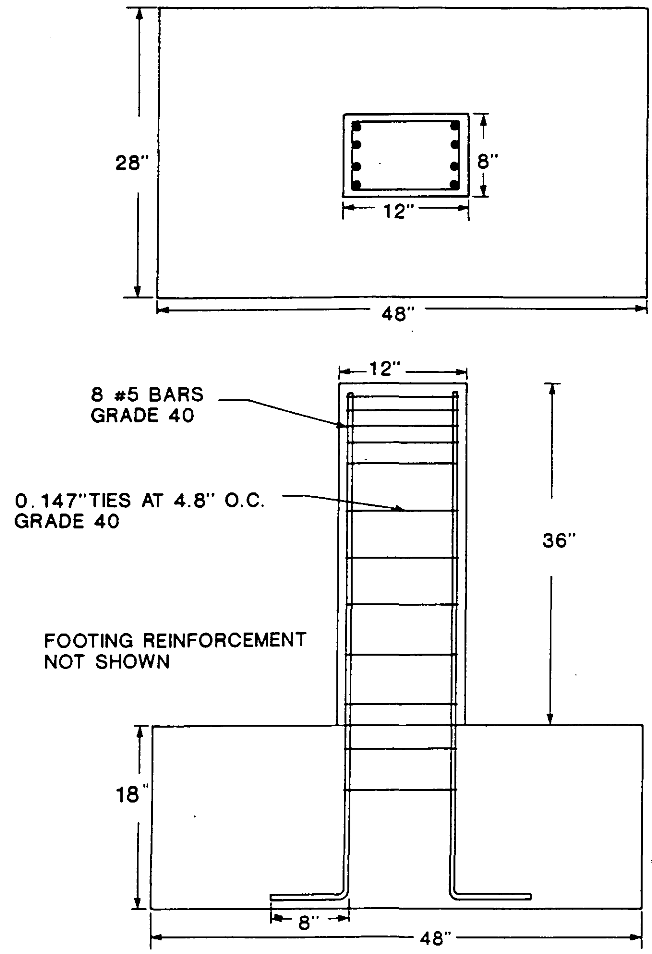

FIGURE 1 Column control specimen: *top*, plan view; *bottom*, elevation view.

Retrofit Methods Studied

Two basic retrofit methods were selected for study, each with a set of parameter variations. The first was a technique using steel plates welded up four longitudinal seams to form a rectangular jacket to encase the full height of the column. It was made slightly oversized for ease in construction, and the gap between this jacket and the specimen was then filled with high-strength, nonshrink cement grout. A ½-in. space was left between the top of the footing and bottom of the jacket to eliminate bearing of the jacket on the footing that would increase the flexural capacity and transfer excessive force to the footing. This retrofit scheme is shown in Figure 2.

The effects of various plate thicknesses in the jacket retrofit method were studied by testing specimens retrofitted with 12- and 16-gauge steel plates, corresponding to approximately ⅒- and 1⁄16-in. thicknesses, respectively. A second variation was to use an epoxy mixture rather than cement grout to fill the gap between the jacket and column. This epoxy mixture is used commercially in anchor bolt applications and consisted of a 1:1 ratio of epoxy to sand in which a rounded sand was used to produce a fairly fluid mixture. A third variation was investigated in which steel dowels were anchored into the column core near the footing to improve confinement of the jacket under cyclic loading and delay longitudinal bar buckling. The dowels used were 0.25-in.-diameter bars set in 4-in.-deep predrilled holes and were anchored with epoxy. They were located at 1 in. and 4 in. from the top of the footing on each 8-in. face.

The second retrofit technique that was investigated used steel angle configurations at each corner of the specimen that were connected by threaded 0.25-in.-diameter rods acting as hoops spaced along the specimen (see Figure 3). These angle/rod configurations, hereafter referred to as retrofit hoops, were expected to act as confinement for the specimen under cyclic loading and to provide shear reinforcement, much like tie reinforcement would in new construction. Expected advantages with this method were the minimal increase in flexural capacity and the ease of applying the retrofit in the field. Various spacings between the hoops were studied.

A summary of the test specimens is given in Table 1.

Test Setup and Procedures

Figure 4 shows the test setup for the column specimens. The specimens were tested using reversed cyclic lateral loading about the strong axis of the section under a constant axial load. Anchor bolts were used to secure the footing to the strong floor, and sliding of the specimen was prevented by horizontal stays. An axial load level of $0.09f'_cA_g$ was applied to all columns, equivalent to a stress level of 360 lb/in.^{2} on the 8- by 12-in. cross section. The axial load varied by at most ±12 percent during testing.

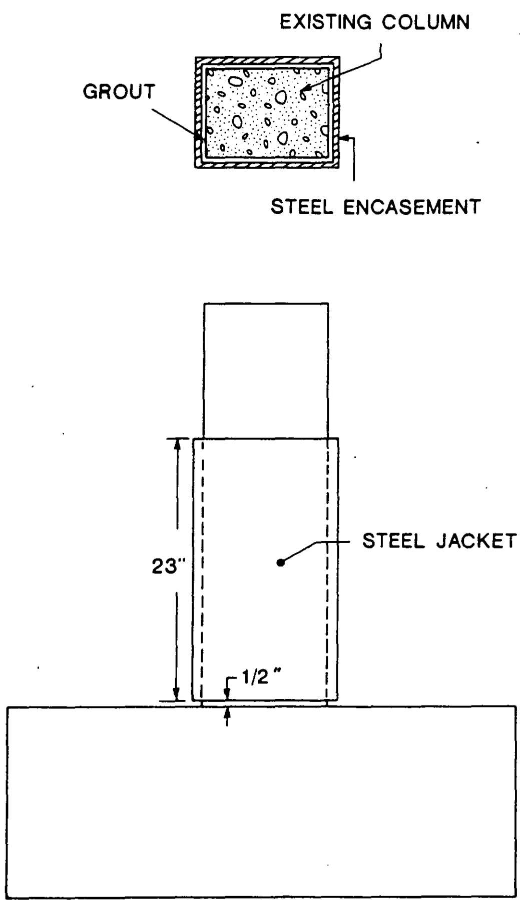

FIGURE 2 Steel jacket retrofit: *top*, plan view; *bottom*, elevation view.

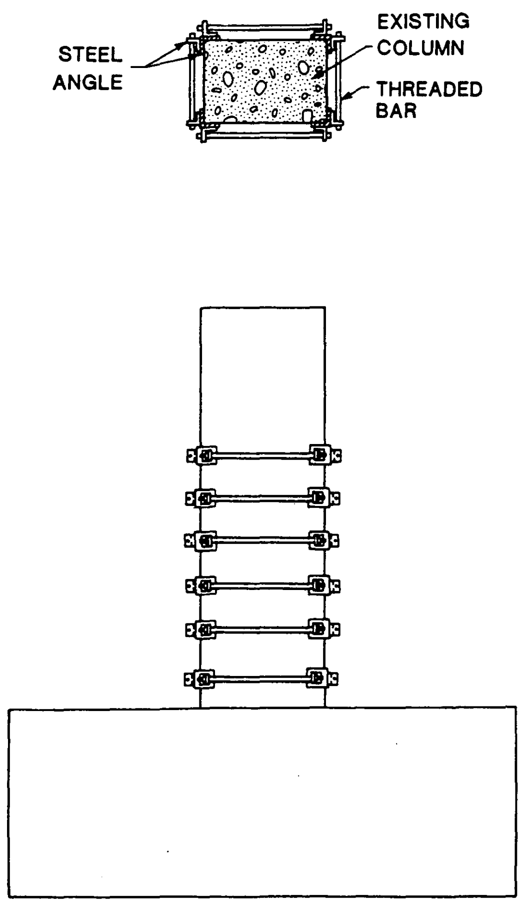

FIGURE 3 Angle and rod retrofit: *top*, plan view; *bottom*, elevation view.

TABLE 1 Column Specimens

SPECIMEN NO.	COLUMN SPECIMEN DESCRIPTION
1	Control
2	Control
3	Angle/rod retrofit, 6 in. o.c.
4	Steel jacket retrofit, 16 gage
5	Angle/rod retrofit, 4 in. o.c.
6	Steel jacket retrofit, 12 gage
7	Steel jacket retrofit, 16 gage with dowels
8	Steel jacket retrofit, 16 gage with epoxy

The cyclic lateral load was applied at 30 in. above the top of the footing by a 55-kip capacity actuator operated under displacement control using a closed-loop servohydraulic system. The sequence used was chosen to display the general hysteretic characteristics and ductility of each specimen and consisted of an increasing displacement pattern to various multiples of the yield displacement of the column. Two cycles were used at each displacement to displacement ductility levels of $\mu = 1, 2, 4, 6$, and 8, unless premature failure of the column occurred.

Determination of the yield displacement, Δ_y, of the column specimens was altered from typical methods [e.g., Priestly and Park (*9*)] because of restrictions associated with the particular columns under study. Use of the conventional method for the shear deficient columns in this research program was not considered feasible because of the probability of a shear failure occurring before ¾ of the flexural capacity was reached. Therefore, an alternate method using an approximate cracked section analysis was used; this method produced a yield displacement of 0.11 in. For the first column test, this value was used; however, it significantly underpredicted the actual yield displacement evident in the experimental data from Column 1. On the basis of this first test, the actual yield displacement was determined to be 0.26 in. This value was used as the Δ_y for all subsequent column tests to subject the control and retrofitted columns to the same displacement history.

Strain gauges were used to monitor the strains of the flexural and transverse reinforcement as well as the external retrofit rods and steel jackets. All data were recorded intermittently throughout the testing sequence.

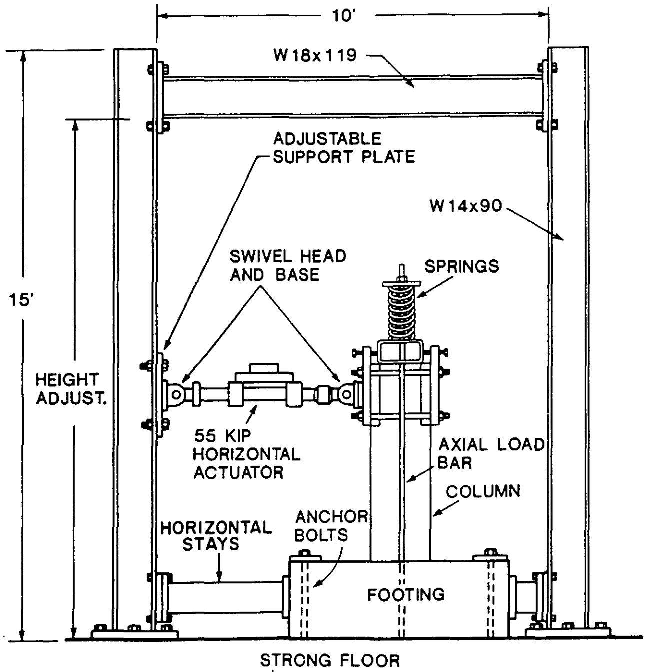

FIGURE 4 Test setup.

TEST RESULTS AND DISCUSSION

In this section, experimental results for the column studies are presented. Results from each specimen are discussed separately to begin with, followed by a discussion of various groups of specimens to facilitate comparisons. Performance of the specimens was mainly evaluated in terms of moment capacity and ductility enhancement along with general hysteretic behavior.

Control Column Test

Column 2, which represented existing field conditions, displayed a classic shear failure with an x-pattern of cracking developing as the testing progressed. The failed control specimen is shown in Figure 5. Results from this column served as the reference for all retrofitted columns. In this specimen, shear cracking and yielding of the ties occurred early on in testing, beginning in the first cycle to 1 Δ_y. Referring to the load-displacement hysteresis shown in Figure 6, there was a sharp degradation in load-carrying capability during the second cycle to 2 Δ_y, after which the load continued to drop until almost no load was carried. This column was evaluated as having a displacement ductility level of $\mu = 1$.

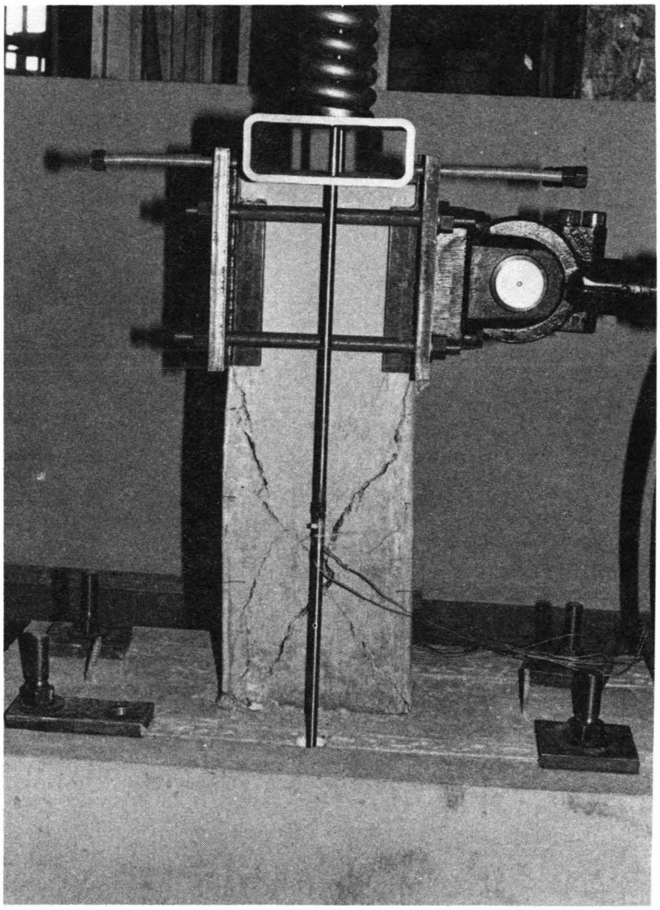

FIGURE 5 Shear cracking in control column specimen.

Angle and Rod Retrofit

Columns 3 and 5 were tested using the angle and rod technique, each with a different spacing between retrofit hoops. For both columns, all bars in the retrofit hoops were uniformly prestressed to 50 percent of the bar's yield stress.

In Column 3, retrofit hoops were used at a spacing of 6 in. on center. During testing, internal tie yielding was observed during the first cycle to 2 Δ_y. At this time displacement level, cracks developed between the corner angles at an inclination of approximately 45 degrees. The load capacity dropped sharply in the first cycle to 4 Δ_y because of brittle fracturing in the threads of the external bars, with failure of the bars beginning at the bottom of the column and working upward. Although a mild steel (A36) was used as the material for the external rods, this was not reflected in the mode of failure. When the retrofit hoops were lost, shear cracks progressively opened up, similar to those observed in the control column. After formation of these large shear cracks, the load capacity continued to drop until almost no load was carried. The lateral load capacity of Column 3 showed an increase of 7 percent over that of the control column because of the addition of the retrofit hoops. A load-displacement hysteresis plot for Column 3 is shown in Figure 7. This column was evaluated as having a ductility of $\mu = 2$.

In Column 5, retrofit hoops were used at a spacing of 4 in. on center. Before this test was conducted, the bars to be used in the retrofit hoops were annealed to produce a material that would respond in a more ductile manner than those used in Column 3. Internal tie yielding did not occur in the testing of Column 5 until the first cycle to 4 Δ_y. Again, shear cracks formed between the corner angles at an inclination of approximately 45 degrees. Column 5, using a smaller spacing

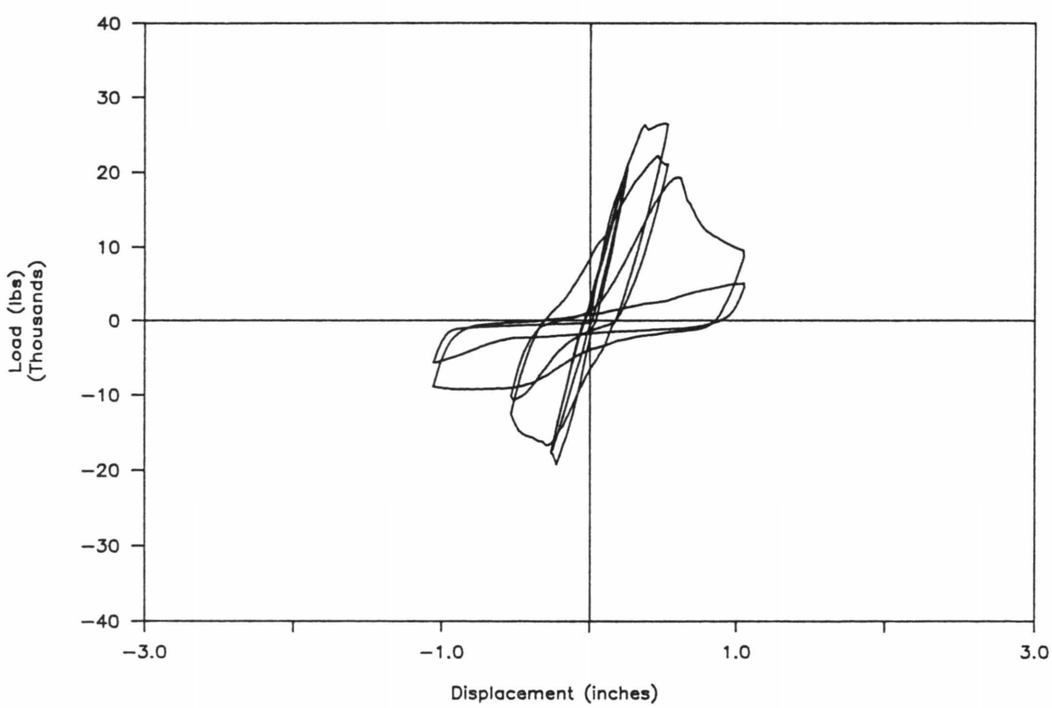

FIGURE 6 Lateral load displacement curves for Specimen 2.

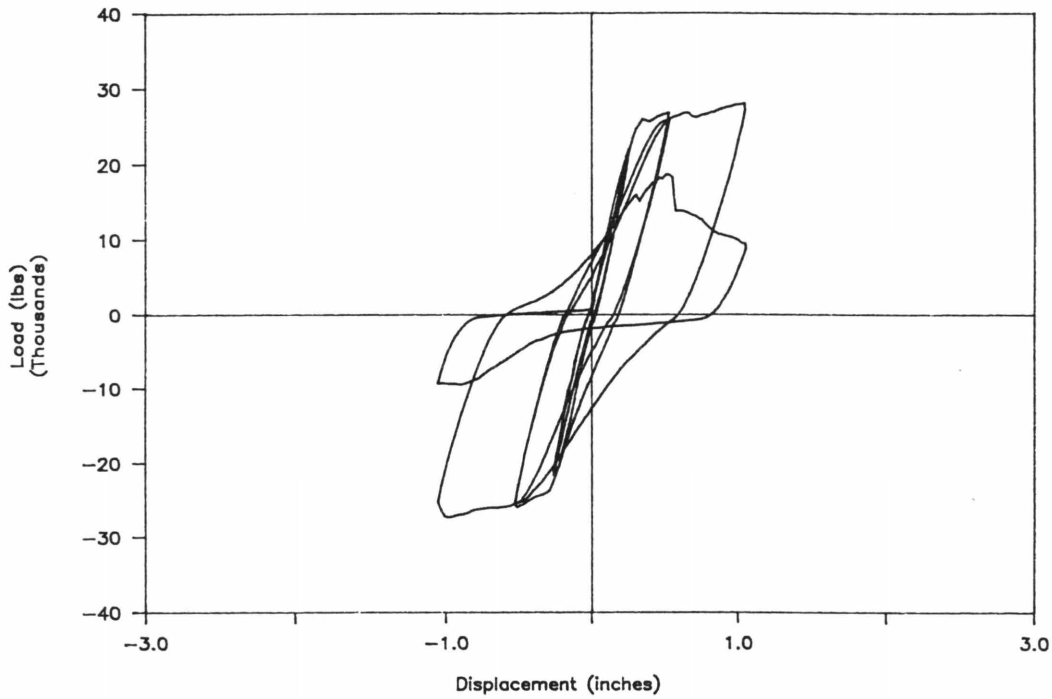

FIGURE 7 Lateral load displacement curves for Specimen 3.

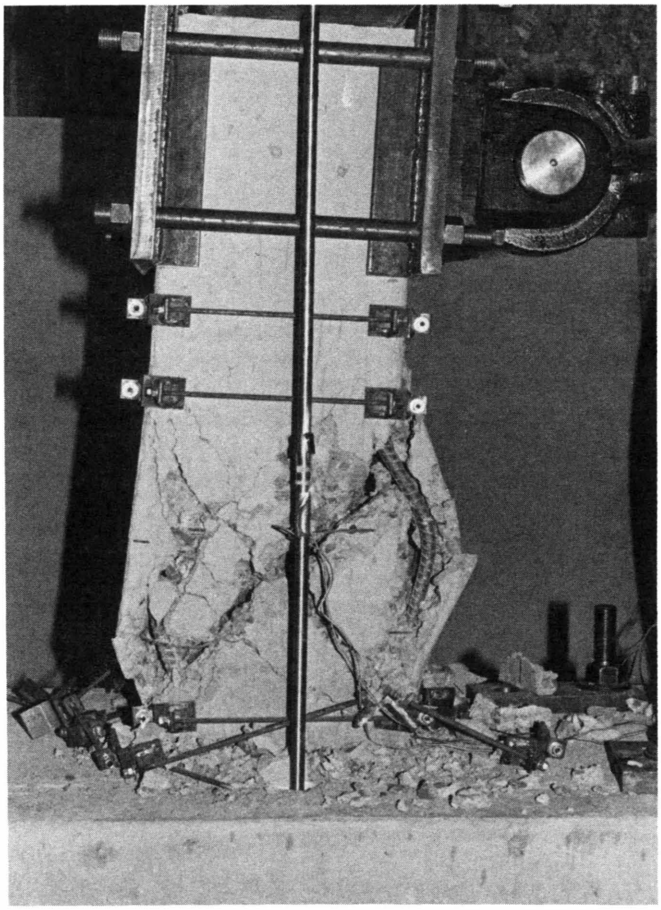

FIGURE 8 Specimen 5 after testing.

between retrofit hoops than in Column 3, held its load into the first cycle to 6 Δ_y, when the external bars began to fail in the threads. However, in this specimen, the bars necked down in a ductile manner before fracturing, with failure of the bars beginning at the middle of the column and working downward. When the retrofit hoops fractured, shear cracks began to open up, as in the control column, followed by a substantial buckling of the longitudinal reinforcement and destruction of the column core (see Figure 8). As with Column 3, the lateral load capacity showed an increase of 7 percent over that of the control column. Column 5 displayed an increase in ductility capacity over that of Column 3, with good performance through $\mu = 4$. A load-displacement hysteresis plot for Column 5 is shown in Figure 9.

This discussion indicates that both columns utilizing the angle and rod retrofit displayed only a moderate amount of improvement in strength and ductility over that of the control column. Column 5, with a smaller spacing between retrofit hoops, resulted in increased ductility capacity and delayed internal tie yielding when compared with Column 3. Both columns showed an increase in lateral load capacity of 7 percent. Of note is the fact that shear cracks developed before fracture of the external hoops, indicating that the mode of failure was not changed from shear to flexure.

With use of this retrofit technique it was expected that the number of retrofit hoops added would increase the shear capacity of the column so that a flexural failure mode would occur. However, the contributions to shear strength from the as-built specimen and the retrofit hoops were not directly additive. This behavior possibly can be explained by limitations in the ductility or elongation of the retrofit bars. As a result, the bars were not able to sufficiently stretch to accommodate the load without fracturing. The use of upset threads

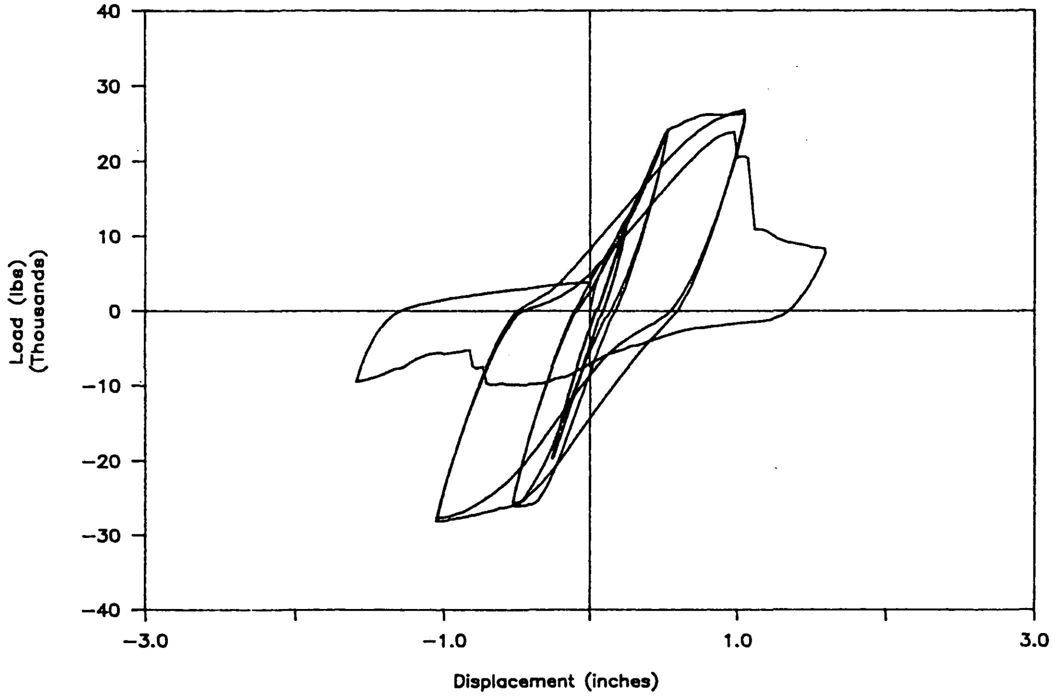

FIGURE 9 Lateral load displacement curves for Specimen 5.

could improve the performance but would significantly increase the cost of the retrofit.

Steel Jacket Retrofit

In the steel jacket retrofit, four columns were tested with variations in the thickness of the steel plate, material in the gap between the existing column and jacket, and confinement near the base of the column.

Column 4, which used a 16-gauge steel jacket with nonshrink cement grout between the existing column and jacket, performed very well under the imposed cyclic loading, with good load-carrying capability through a displacement ductility of $\mu = 8$. A lateral load capacity enhancement of 16 percent was seen in this column when compared with the as-built specimen. From the load-displacement hysteresis plot of Column 4 (Figure 10), it can be seen that the retrofitted column displayed good energy dissipation, and its performance was vastly improved over that of the unretrofitted column. Beginning during cycles to 4 Δ_y, the column longitudinal bars and steel jacket buckled near the base of the column at the maximum moment section. During testing to larger displacement levels, buckling increased, but the jacket continued to provide some confinement to the hinging region. As a result of this confinement, cracks penetrated into the footing because the plastic hinging region was forced downward into the footing. Throughout the testing sequence, there was no evidence of internal tie yielding. After the jacket had been removed, no evidence of any shear cracks was visible in the column.

Column 6, which was retrofitted using a 12-gauge steel jacket and cement grout between the jacket and existing column, also performed well with good load-carrying capability through a ductility of $\mu = 8$. Even though the steel jacket in Column 6 was 75 percent thicker than that in Column 4, the lateral load capacities were the same. From the load-displacement hysteresis plot of Column 6 shown in Figure 11, the width of the loops, which are slightly wider than those in Column 4, indicates good energy dissipation throughout the test sequence. As in Column 4, buckling of the longitudinal bars and steel jacket occurred near the base of the column beginning during the cycles to 4 Δ_y. However, the extent of buckling was slightly reduced by the use of a thicker jacket. Cracking was again seen in the footing around the base of the column because of the downward shifting of the plastic hinge zone. Internal tie yielding was prevented using this retrofit scheme, and no shear cracks were seen in the column after the jacket had been removed.

Column 7, which used a 16-gauge steel jacket, cement grout, and four steel dowels into the column core, showed excellent performance through a ductility of $\mu = 8$. This specimen had a lateral load capacity increase of 19 percent over that of the control column, which was the largest of all columns tested, possibly because of the increased confinement at the plastic hinge. In addition, the use of dowels through the column core near the maximum moment section resulted in an increase in energy dissipation, as seen by the width of the loops in the load-displacement hysteresis plot in Figure 12. These hysteresis loops were the widest of all columns tested. By using dowels through the column core, buckling on one side of the column was essentially eliminated. However, buckling was seen on the other side of Column 7, possibly because of the severing of an internal tie while drilling into the core on this side of the column. As with the other jacketed columns, no shear cracks were seen after removal of the steel jacket, and

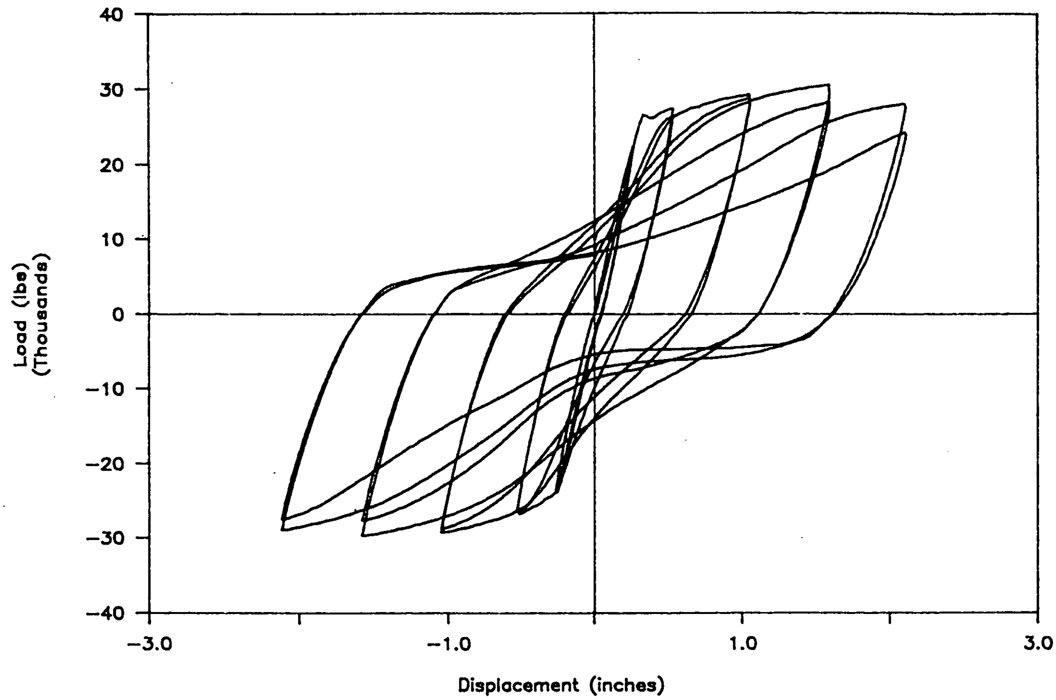

FIGURE 10 Lateral load displacement curves for Specimen 4.

cracking in the footing was seen because of the downward migration of the zone associated with plastic hinging.

In Column 8, which used a 16-gauge steel jacket with epoxy between the jacket and column, very good load-carrying capability was seen though $\mu = 8$. This retrofit resulted in a lateral load capacity increase of 18 percent over that of the control column. In this specimen, tie yielding was prevented throughout the testing sequence, and no shear cracks were seen after the steel jacket had been removed. The load-displacement hysteresis curves of Column 8 (Figure 13) are almost identical to those of Column 6 with the thicker jacket. With the epoxy filler, performance is slightly improved over that in Column 4, which used the cement grout, but not enough to justify the substantially higher cost of the epoxy. The use

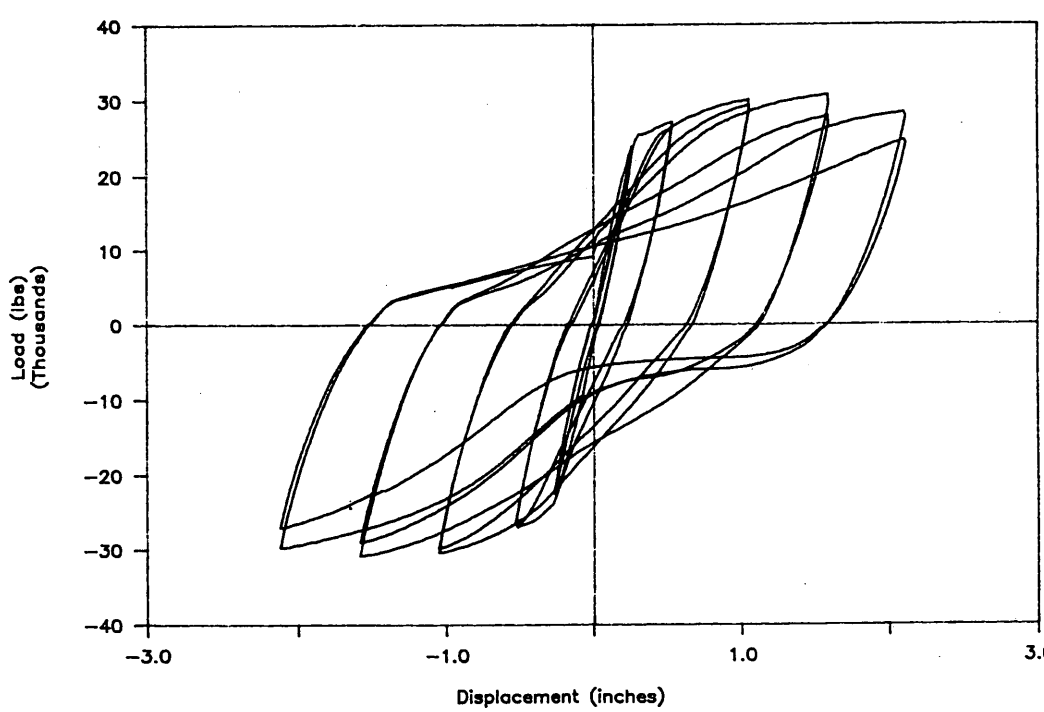

FIGURE 11 Lateral load displacement curves for Specimen 6.

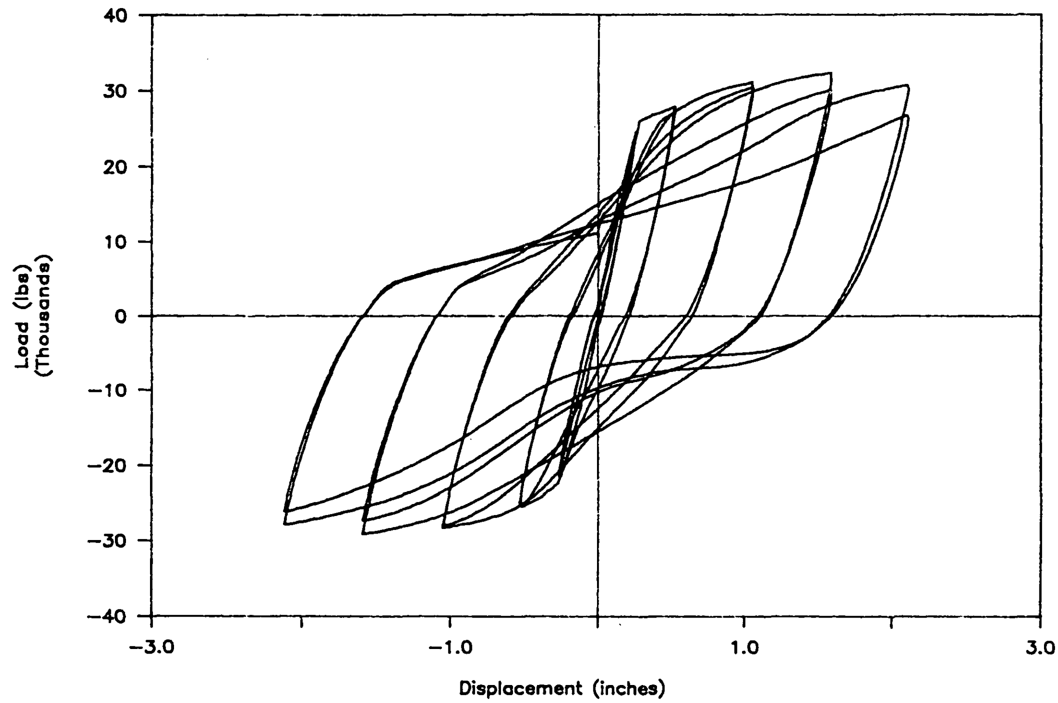

FIGURE 12 Lateral load displacement curves for Specimen 7.

of epoxy between the existing column and jacket significantly delayed buckling of the longitudinal reinforcement and steel jacket, but buckling ultimately reached the same extent as that in Column 6. Cracking of the footing due to shifting of the plastic hinge zone also occurred.

From this discussion it can be seen that all steel-jacketed specimens performed well in improving the strength and ductility capacity of the deficient section. In each column, a ductility of $\mu = 8$ was achieved with good load-carrying capability, after which testing was stopped. Load capacity increases were nearly uniform for all jacketed specimens, ranging from 16 to 19 percent, with the highest value in the specimen incorporating dowels through the column core. In all jacketed columns, initial stiffnesses were only slightly larger than that

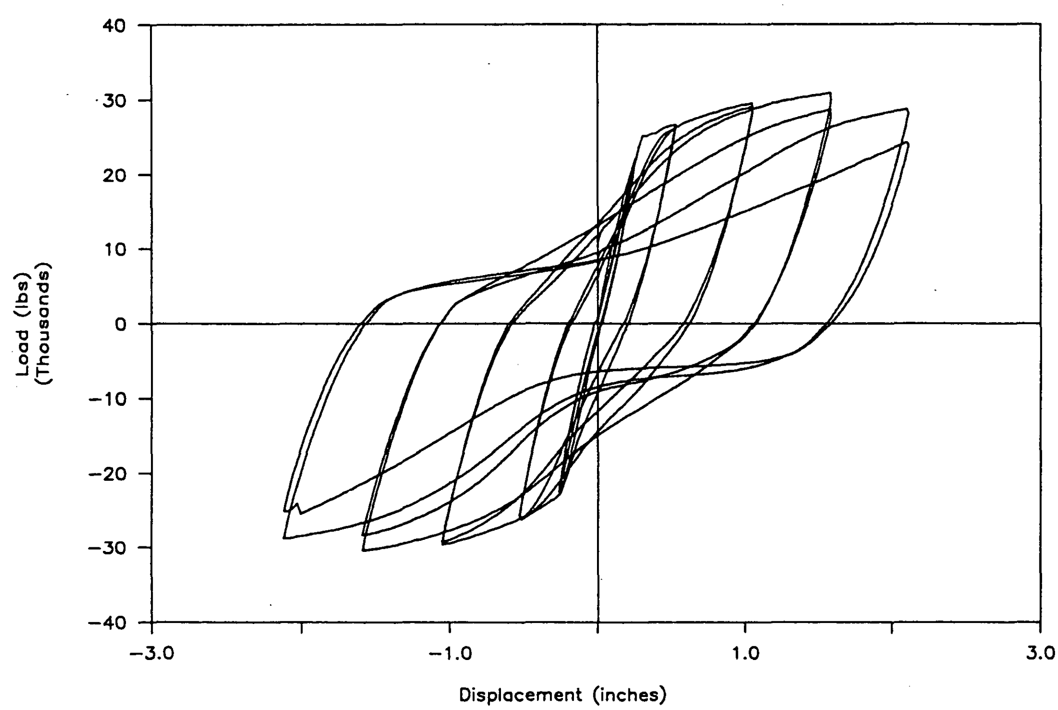

FIGURE 13 Lateral load displacement curves for Specimen 8.

for the control column. Yielding of the internal ties was prevented using this retrofit method. After the steel jackets had been removed at the completion of testing, no shear cracks were seen in any of the columns, indicating that the mode of failure was changed from shear to flexure. The energy absorption characteristics of the jacketed specimens were improved substantially when compared with those seen in Column 2. The use of dowels through the column core at the maximum moment section resulted in the most energy dissipation of all specimens. Increasing the thickness of the steel jacket slightly improved the energy dissipation, as did using the epoxy mixture instead of a nonshrink grout between the column and steel jacket.

Throughout the testing sequence, each steel-jacketed specimen showed some buckling of the longitudinal reinforcement and jacket at the base of the column under the imposed cyclic loading. By using dowels through the column core, buckling was reduced and on one side was essentially eliminated. Although the jackets yielded because of buckling of the longitudinal bars, all jacketed columns maintained some confinement in the hinging region. However, tests by Chai et al. (*3*) have shown that this level of confinement provided by the buckled rectangular steel jacket would be insufficient to prevent strength degradation if an inadequate longitudinal reinforcement splice was present in the plastic hinging region.

CONCLUSIONS

On the basis of the results of this experimental investigation, both the angle and rod retrofits and the steel jacket retrofit were beneficial in improving the seismic performance of shear-deficient rectangular bridge columns. Only moderate improvements in ductility resulted from the application of the angle and rod retrofit. The full-height rectangular steel jacket retrofit provided significant enhancement in lateral load capacity and ductility of the deficient section. Although buckling of the steel jacket at the maximum moment section occurred, sufficient confinement was provided to maintain stable hinging behavior. This buckling was reduced, and specimen performance was improved, by providing dowels through the column core in the maximum moment region. An elliptical or circular shaped jacket used locally in the hinging regions would improve confinement and should be investigated in future research.

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