Seismic Retrofitting of Rectangular Bridge Column for Shear

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16. ABSTRACT

This study investigated retrofitting measures applied to 2/5-scale shear-deficient columns representative of rectangular bridge columns in the Puget Sound area of Washington state. 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. The performance of the specimens was evaluated in terms of load capacity and ductility.

Tests on the as-built column resulted in a brittle shear failure at the calculated yield displacement, i.e., at a displacement ductility level of $\mu = 1$. Both retrofit methods 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, and displacement ductility levels of $\mu = 2$ and 4 were achieved.

With 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 $\mu = 8$. When this retrofit was applied over the full height of the column, the steel jacket increased the column shear strength enough that flexural failure 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.

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Seismic Retrofit of Bridge Columns

SEISMIC RETROFITTING OF RECTANGULAR BRIDGE COLUMN FOR SHEAR

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DISCLAIMER

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SUMMARY

This study investigated retrofitting measures applied to 2/5-scale shear deficient columns representative of rectangular bridge columns in the Puget Sound area of Washington state. 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. The performance of the specimens was evaluated in terms of load capacity and ductility.

Tests on the as-built column resulted in a brittle shear failure at the calculated yield displacement, i.e., at a displacement ductility level of $\mu = 1$. Both retrofit methods improved the behavior of the deficient column.

The external hoop retrofit improved the performance of the retrofitted columns only moderately over that of the as-built column. Brittle fracture of the retrofit hoops limited the load-carrying capability and ductility enhancement, and displacement ductility levels of $\mu = 2$ and 4 were achieved.

The rectangular steel jacket retrofit improved the column's performance significantly over that of the as-built column. The jacket retrofit produced a ductile column response with good load-carrying capability through $\mu = 8$. When this retrofit was applied over the full height of the column, the steel jacket increased the column shear strength enough that flexural failure 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.

CONLUSIONS AND RECOMMENDATIONS

On the basis of the results of this experimental investigation, the following conclusions and recommendations are made.

1. The as-built column specimen exhibited poor ductility under the imposed reversed cyclic loading. The tests on this column produced brittle shear failure at a displacement level that corresponded to yielding of the flexural reinforcement in the column, i.e., at a displacement ductility level of $\mu = 1$.

2. In the columns retrofitted with the external hoop retrofit, ductility and strength were only moderately improved over those of the as-built column because of brittle fracture of the retrofit hoops. Displacement ductility levels of $\mu = 2$ and 4 and capacity increases of 7 percent over the as-built column were achieved in the tests on these columns. Shear cracking was observed in the columns even before the loss of any of the retrofit hoops, indicating that shear was the mode of failure in these retrofitted columns. The contributions to shear strength from the as-built column and the retrofit hoops were not additive.

3. Rectangular steel jacket retrofit improved the column's performance significantly over that of the as-built column. Displacement ductility levels of $\mu = 8$ and capacity increases of approximately 17 percent over the as-built column were achieved in all tests on the jacketed column specimens. No tie yielding or shear cracking were observed in the columns, indicating that flexure was the mode of failure in these retrofitted columns.

4. Buckling of the steel jacket and longitudinal reinforcement occurred near the maximum moment section in the columns retrofitted with the steel jacketing. However, the buckled steel jacket provided sufficient

confinement to the hinging region to maintain load-carrying capability. If a longitudinal reinforcement splice had been present in the hinging regions of a column, it is unlikely that the level of confinement provided by the buckled steel jacket would have been adequate to prevent strength degradation.

5. To reduce the buckling of the steel jacketing observed in the specimens, the rectangular jacketing should be modified in the hinging regions to increase confinement. The results of this study indicated that dowels in the column core increased the confinement in the hinging region. Future research should investigate an increased thickness of steel jacket or an elliptical or circular shaped jacket used locally in the hinging regions.

6. The use of epoxy grout between the steel jacketing and existing column improved performance only slightly over a column that had used non-shrink cement grout. Therefore, cement grout should be used because of the significantly higher cost associated with the epoxy grout.

INTRODUCTION

RESEARCH OBJECTIVES

The objectives of this study were as follows:

1. to identify deficient reinforcing details that may result in shear failures and low ductility levels in existing bridge columns under seismic loading;
2. to identify retrofit techniques for increasing the shear strength and ductility capacity of the bridge columns;
3. to experimentally evaluate the benefits of several techniques for retrofitting rectangular bridge columns for shear; and
4. to draw conclusions on the effectiveness of the shear retrofit methods studied and to identify areas where further research is needed.

THE PROBLEM

The extensive damage to bridge structures in the 1971 San Fernando earthquake caused engineers to significantly re-evaluate the seismic design of bridges. Since then, many improvements have been incorporated into updated design criteria. However, many bridges still in use 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 have clearly indicated the vulnerability of older bridges and the need to develop methods for strengthening these bridges to meet current safety requirements.

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

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 were anchored only with lapped ends in the cover concrete. Further, intermediate ties were rarely used. This detail means that many older columns are susceptible to shear failures, and little confinement is available to develop full flexural capacity or prevent buckling of the longitudinal reinforcement.

BRIDGE COLUMN RETROFITTING

PREVIOUS RETROFIT RESEARCH

Chai, Priestley, and Seible (1, 2) examined the effectiveness of retrofitting circular bridge columns with circular steel jackets and rectangular bridge columns with elliptical steel jackets, filling the gap between the jacket and column with high-strength grout. Initially, the jacket was used only in the plastic hinge region and terminated just above the footing. Unretrofitted circular columns with lapped starter bars did not reach their theoretical strength because of bond failure in the early stages, after which their 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, bond failure at the splice led to rapid strength and stiffness degradation. When they were retrofitted with a 3/16-in. thick elliptical jacket, excellent hysteretic response resulted. A later phase of testing showed that the same steel jackets, extended over the full height of the same types of columns, were effective in enhancing shear strength and ductility.

Coffman, Marsh and Brown (3) studied a retrofit method for improving bridge column ductility that placed external hoops, prestressed with turnbuckles, around the lower portion of circular columns. This scheme dramatically increased the total energy dissipation of the section and increased 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, Klingner and Jirsa (4, 5) improved the performance of rectangular columns by adding external longitudinal reinforcement and closely-spaced ties. They varied this retrofit method by adding crossties through the column, which were anchored by hooking around the longitudinal reinforcement. The crossties improved the confinement, decreasing strength and stiffness degradation under reversed cyclic loading.

Fyfe and Priestley (6) studied a retrofit method that utilized a high-strength, fiber-reinforced fabric that was post-tensioned around the plastic hinging region of a column. This retrofit method enhanced flexural ductility and prevented the bond failures that were observed in the as-built columns tested by Priestley et al. (1, 2).

CURRENT RETROFIT PRACTICE

Currently, little information exists on standard procedures for retrofitting bridge substructures. In the U.S., only Caltrans has implemented any standardized procedures for selecting critical substructure elements and specifications for retrofitting them once a bridge substructure has been identified as critical (7).

In bridges with columns identified as unsatisfactory, Caltrans has standardized two column retrofit methods, the Class P retrofit and the Class F retrofit. These involve the placement of 3/8-in. thick steel jackets around the columns. For shear-deficient columns, the jacket is full height. 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. 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-in. by 30-in. section, with reinforcement concentrated on the 20-in. faces and 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 2/5-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. The footing was designed to be stronger than those commonly found in pre-1971 designs to prevent a footing failure that would introduce another variable into the testing program. Continuous longitudinal bars extended into the footing, with 90° hooks for resistance to pullout, as shown in Figure 2. 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 seen.

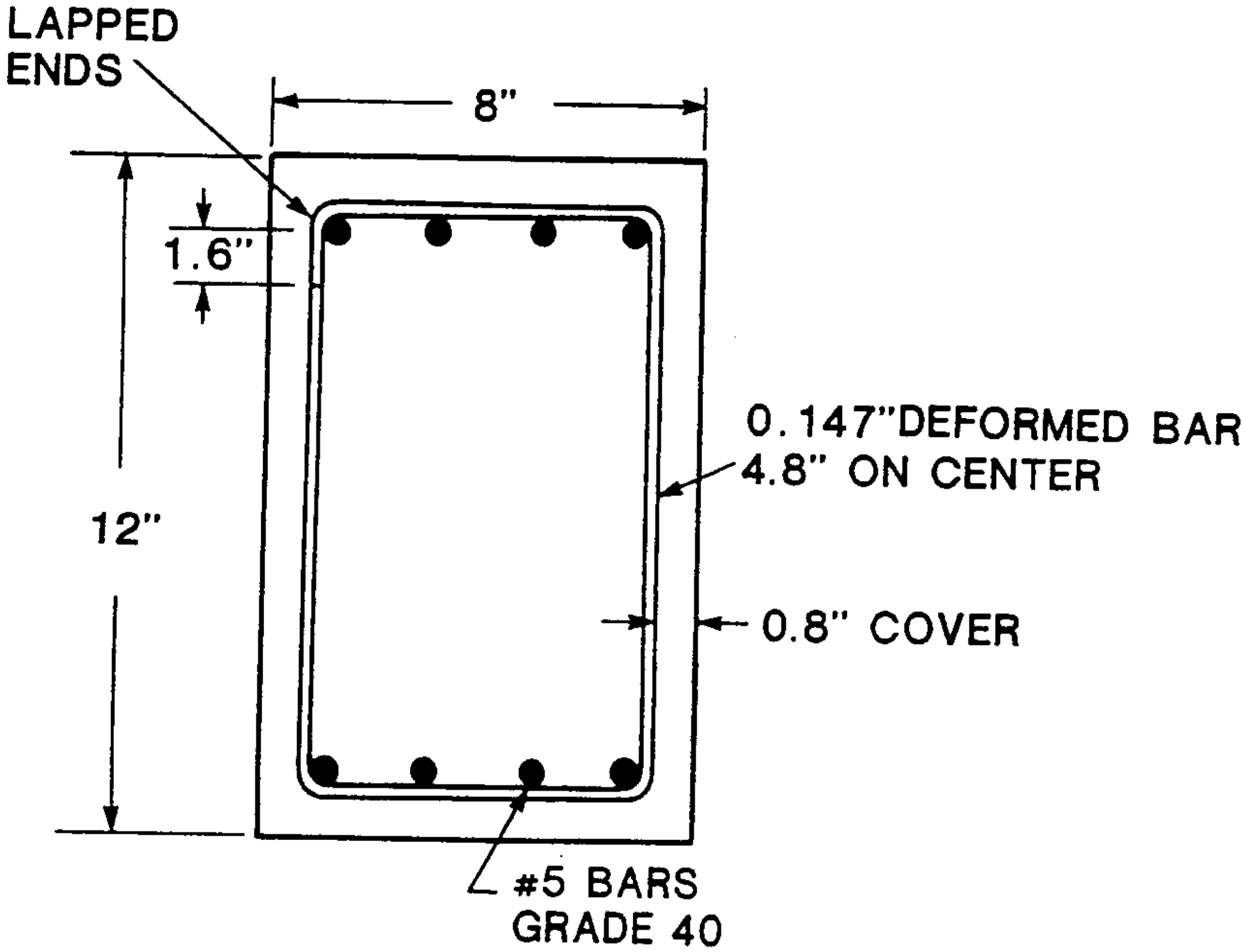

Figure 1. Cross-Section of the Control Column Specimen

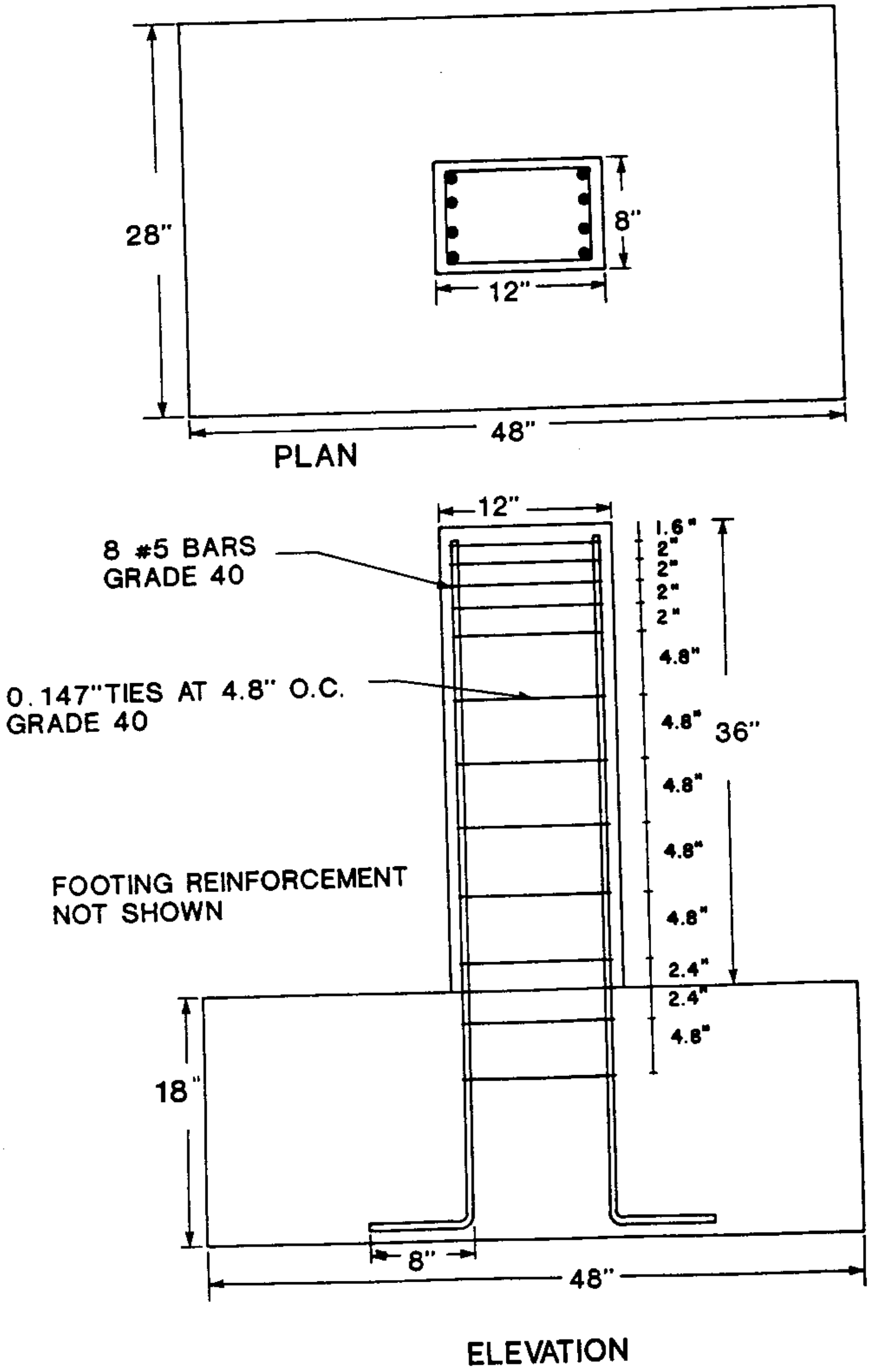

Figure 2. Elevation and Plan Views of Control Column Specimen

RETROFIT METHODS STUDIED

Two retrofit methods were selected for study, each with a set of parameter variations. The first was a technique that used steel plates welded up four longitudinal seams to form a rectangular jacket to encase the full height of the column. The jacket was made slightly oversized for ease in construction, and the gap between this jacket and the specimen was then filled with high-strength, non-shrink cement grout. A 1/2-in. space was left between the top of the footing and the bottom of the jacket to prevent the jacket from bearing on the footing, which would have increased the flexural capacity and transferred excessive force to the footing. Similarly, a space was left between the top of the jacket and the loading collar. This retrofit scheme is shown in Figure 3.

The effects of different plate thicknesses in the jacket retrofit method were studied by testing specimens retrofitted with 12-gage and with 16-gage steel plates, which corresponded to approximately 1/10-in. and 1/16-in. thicknesses, respectively. A second variation involved the use of an epoxy mixture to fill the gap between the jacket and column, instead of cement grout. This epoxy mixture is used commercially in anchor bolt applications and consisted of 1:1 ratio of epoxy to sand. A rounded sand was used to produce a fairly fluid mixture. In a third variation, steel dowels were anchored into the column core near the footing to improve confinement of the jacket under cyclic loading and to delay longitudinal bar buckling. The dowels were 0.25-in. diameter bars set in 4-in. deep, pre-drilled holes, and they were anchored with epoxy. The ends of the dowels were threaded, and washers and nuts were connected to the dowels in order to restrain buckling of the jacket, as shown in Figure 4. The dowels were located at 1 in. and 4 in. from the top of the footing on each 8 in. face.

In the second retrofit technique, steel angle configurations at each corner of the specimen were connected by threaded, 0.25-in. diameter rods that acted as hoops spaced along the specimen. This scheme is shown in Figure 5. These angle/rod configurations,

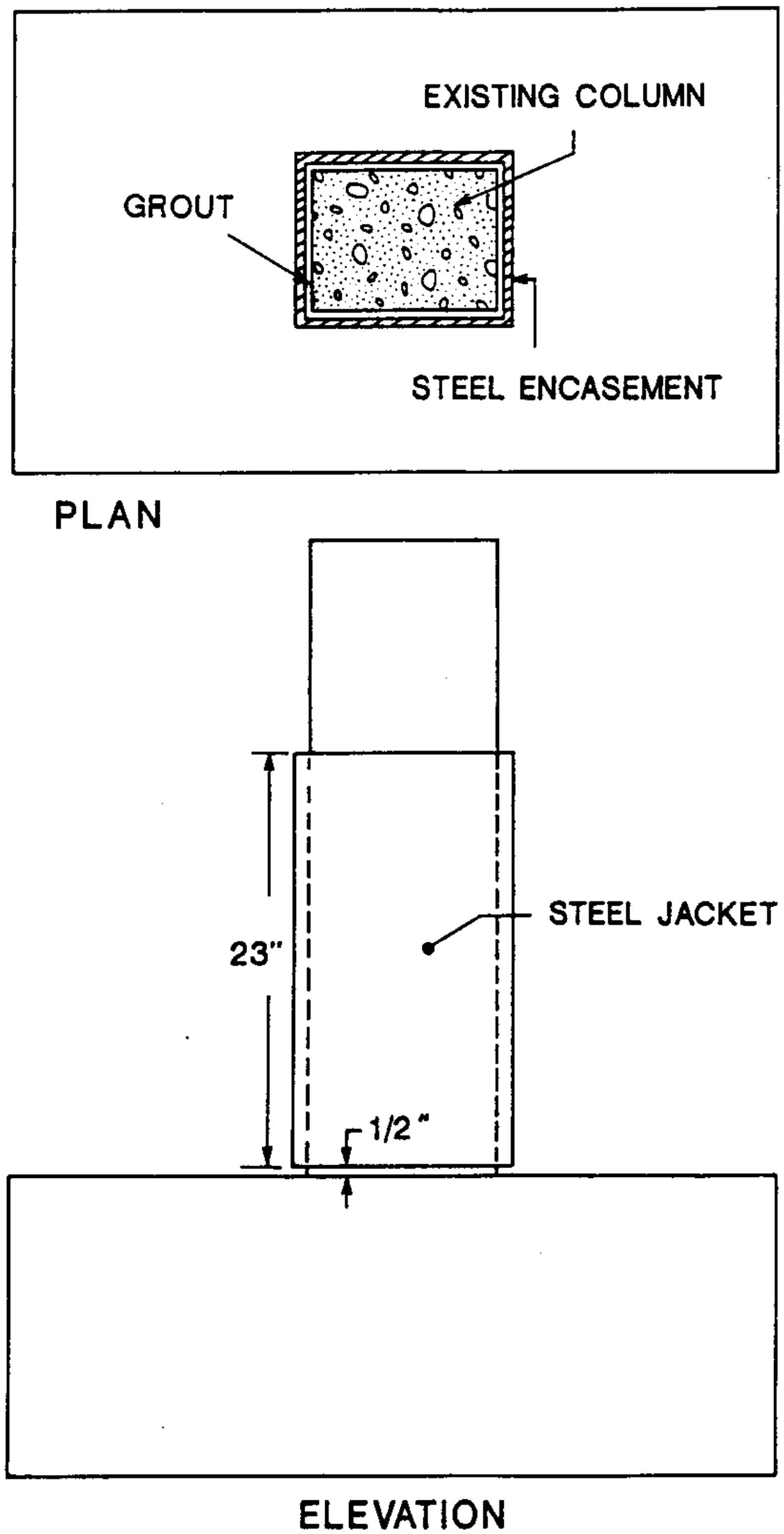

Figure 3. Steel Jacket Retrofit

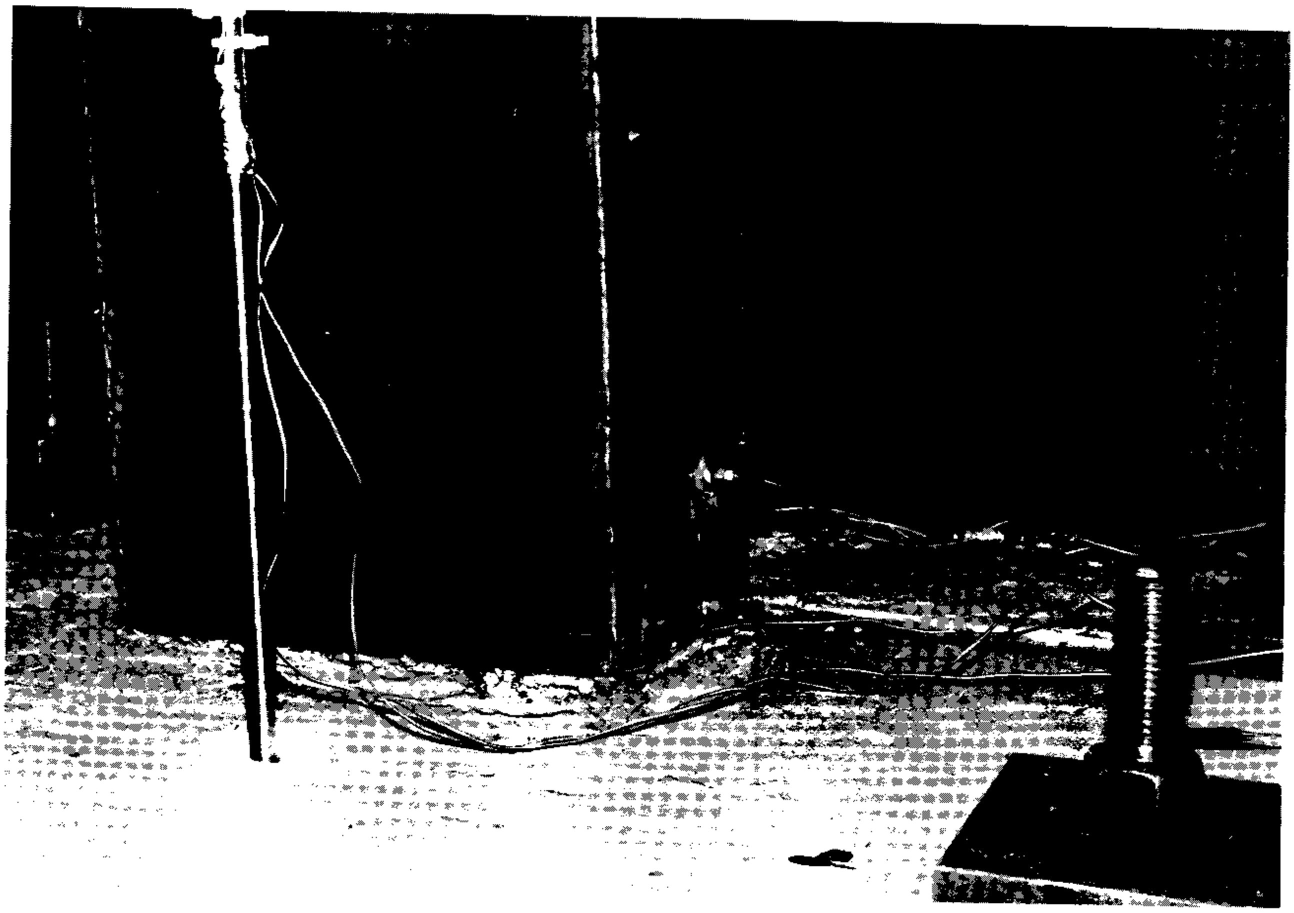

Figure 4. Dowels Used to Restrain Jacket Buckling in the Hinge Region

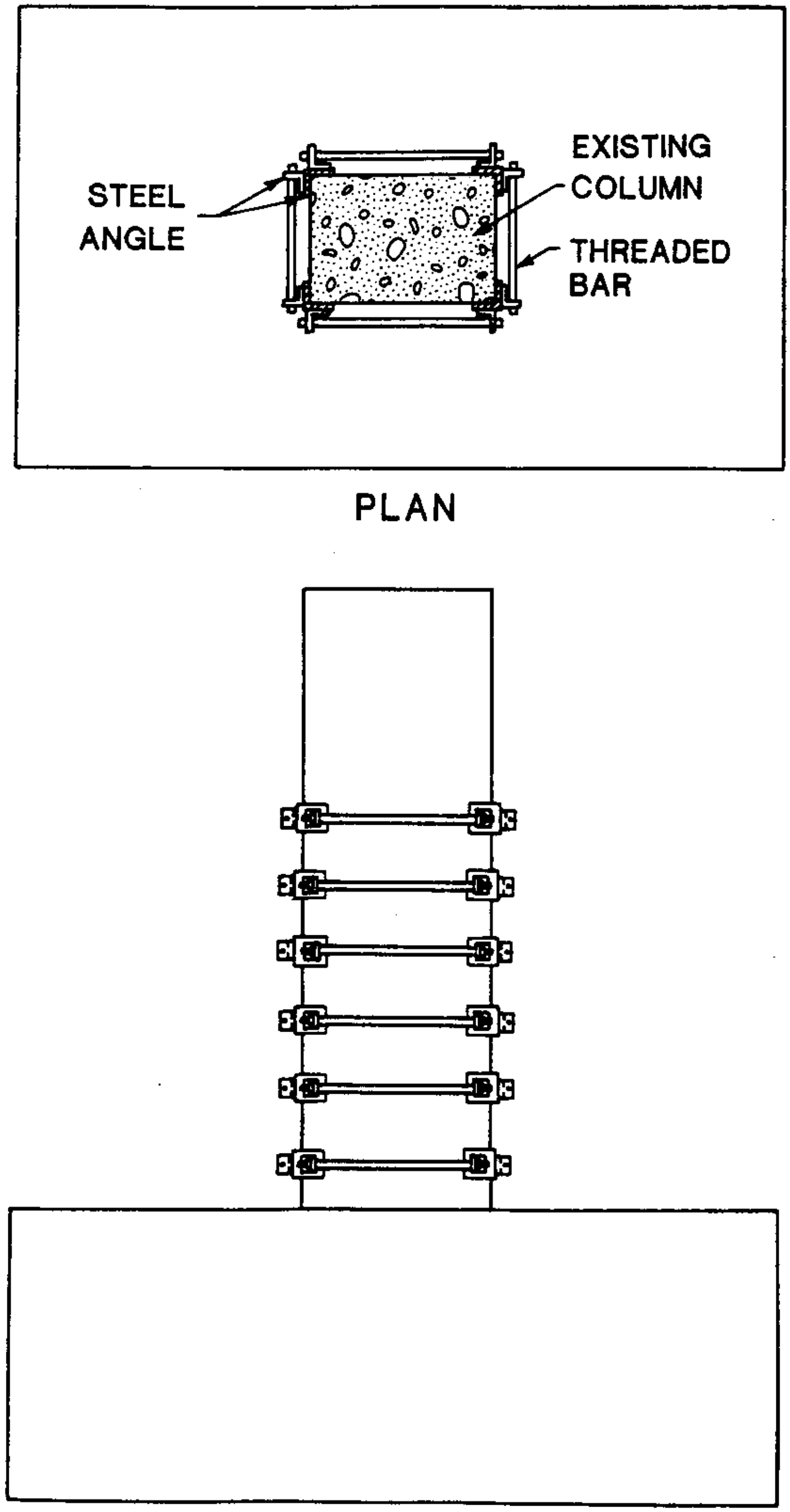

Figure 5. Angle and Rod Hoop Retrofit

hereafter referred to as retrofit hoops, were expected to confine the specimen under cyclic loading and to provide shear reinforcement, much like tie reinforcement would in new construction. The expected advantages with this method were the minimal increase in flexural capacity and the ease of applying the retrofit in the field. Different spacings between the hoops were studied.

Summaries of the test specimen descriptions and material strengths are given in tables 1 and 2, respectively. Additional details of the testing program can be found in Reference 8.

Table 1. Description of 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

Table 2. Material Strengths

Concrete compressive strength	3900 psi
Cement grout compressive strength	6900 psi
Epoxy grout compressive strength	9400 psi
Longitudinal reinforcement yield strength	48,000 psi
Transverse reinforcement yield strength	41,000 psi
Steel jacket yield strength	43,000 psi
Retrofit bar yield strength	65,000 psi
Retrofit bar yield strength (annealed)	53,000 psi

TEST SETUP AND PROCEDURES

Figure 6 shows the test setup for the column specimens. The specimens were tested with reversed cyclic lateral loading about the strong axis of the section under a constant axial load. Anchor bolts secured the footing to the strong floor and the specimen was prevented from sliding with horizontal stays. The axial load was delivered by a spring system in which two 5/8-in. diameter high-strength bars with threaded ends were passed through the footing and anchored to the strong floor. The researchers stressed these bars by jacking them against two large-capacity steel springs at the top of the column. An axial load level of $0.09f'_cA_g$, equivalent to a stress level of 360 psi on the 8-in. by 12-in. cross-section, was applied to all columns. The axial load varied by at most ±12 percent during testing.

The cyclic lateral load was applied 30 in. above the top of the footing by a 55-kip capacity actuator operated under displacement control with a closed loop, servohydraulic system. The load sequence was chosen to display the general hysteretic characteristics and ductility of each specimen and consisted of a displacement pattern that increased to various multiples of the yield displacement of the flexural reinforcement. Two cycles were used at each displacement to structure ductility levels of μ = 1, 2, 4, 6, and 8, as can be seen in Figure 7, unless premature failure of the column occurred.

The method for determining the yield displacement, Δ_y, of the column specimens was altered from typical methods (e.g., Priestley 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 considered unfeasible because of the probability that shear failure would occur before 3/4 of the flexural capacity had been reached. Therefore, the researchers used an alternate method that involved an approximate cracked section analysis, which produced a yield displacement of 0.11 in. For the first column test, this value was used; however, it significantly underpredicted the

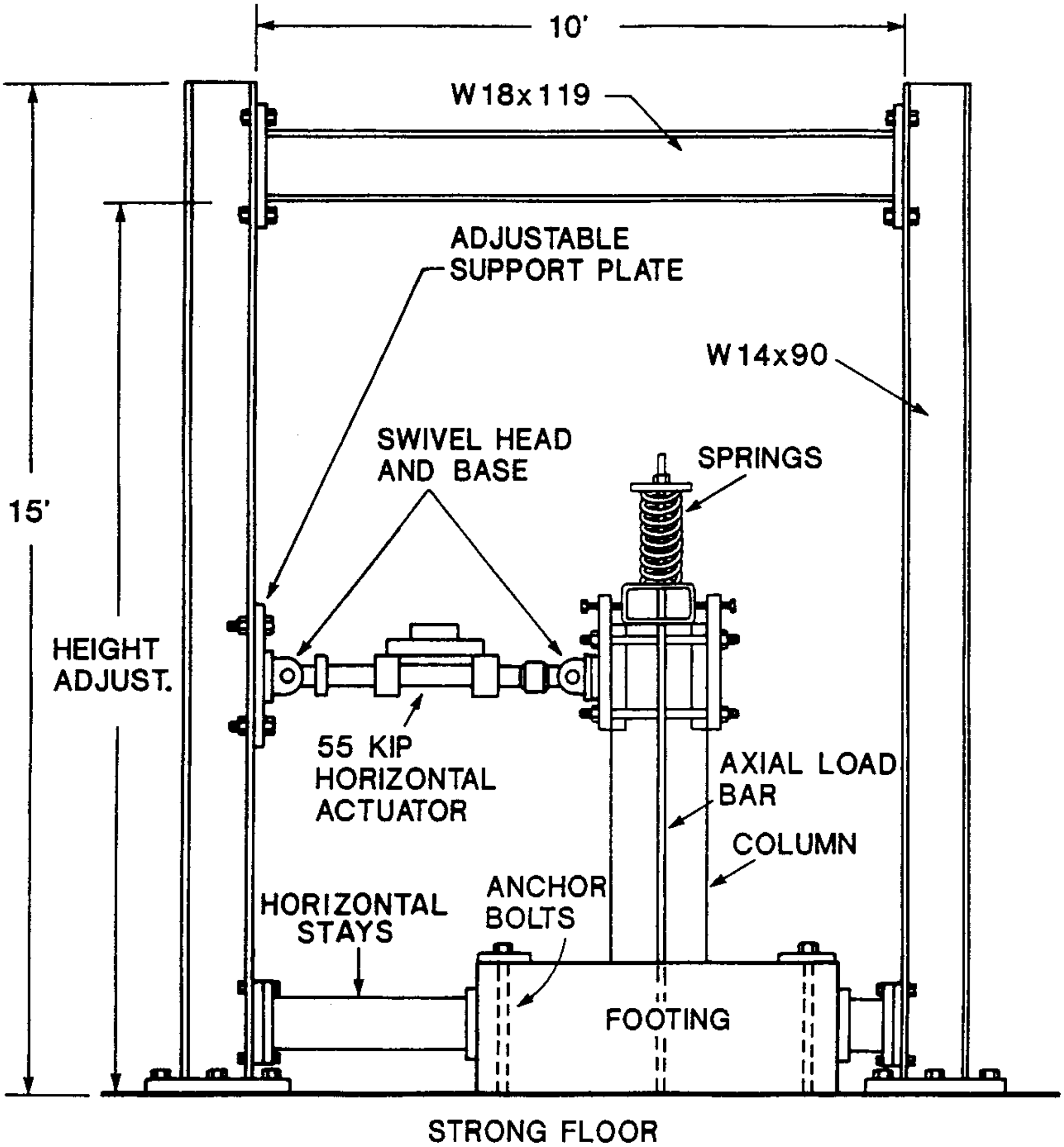

Figure 6. Test Setup

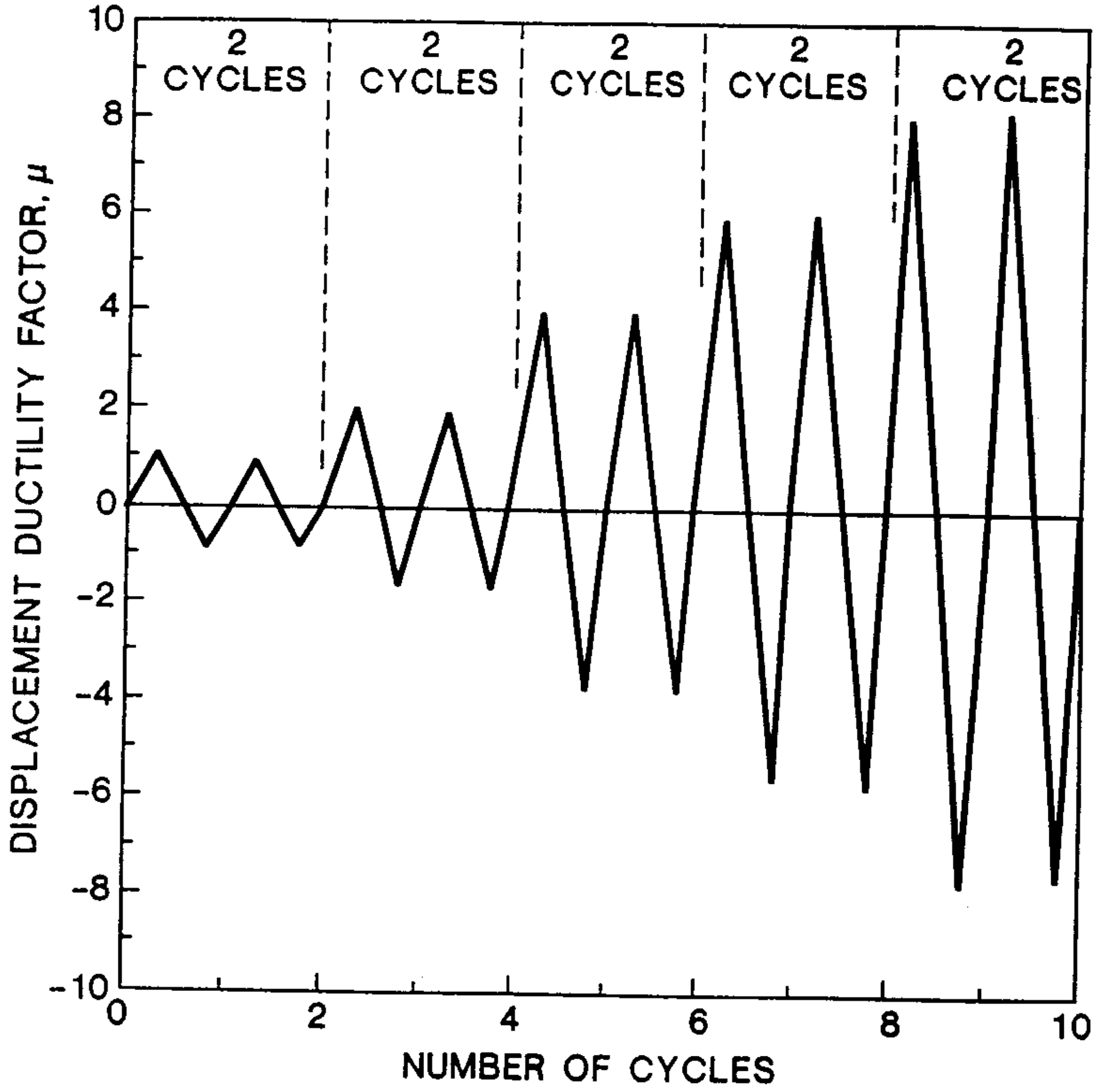

Figure 7. Lateral Loading Sequence

yield displacement evident in the experimental data from column No. 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 so that the control and retrofitted columns would be subjected to the same displacement history.

Strain gages monitored 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.

TEST RESULTS AND DISCUSSION

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

CONTROL COLUMN TESTS

Both column Nos. 1 and 2, which represented existing field conditions, displayed classic shear failures in which an x-pattern of cracking developed as the testing progressed. A failed control specimen is shown in Figure 8.

In column No. 1, the lateral loading sequence imposed was based on a yield displacement of 0.11 in., which substantially underpredicted the actual Δ_y, as discussed earlier. Therefore, the results from this test were not valid for evaluating the ductility capacity and general hysteretic characteristics of the column. However, from these data the actual yield displacement was extracted and used in subsequent tests.

Column test No. 2 was subjected to a lateral loading sequence that was based on a yield displacement of 0.26 in., which was determined from the data collected in the first column test. The results from this column served as the reference for all retrofitted columns. In this specimen, shear cracking and yielding of the ties occurred early in testing, beginning in the first cycle to 1 Δ_y. The load-displacement hysteresis in Figure 9 shows 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 determined to have a displacement ductility level of $\mu = 1$.

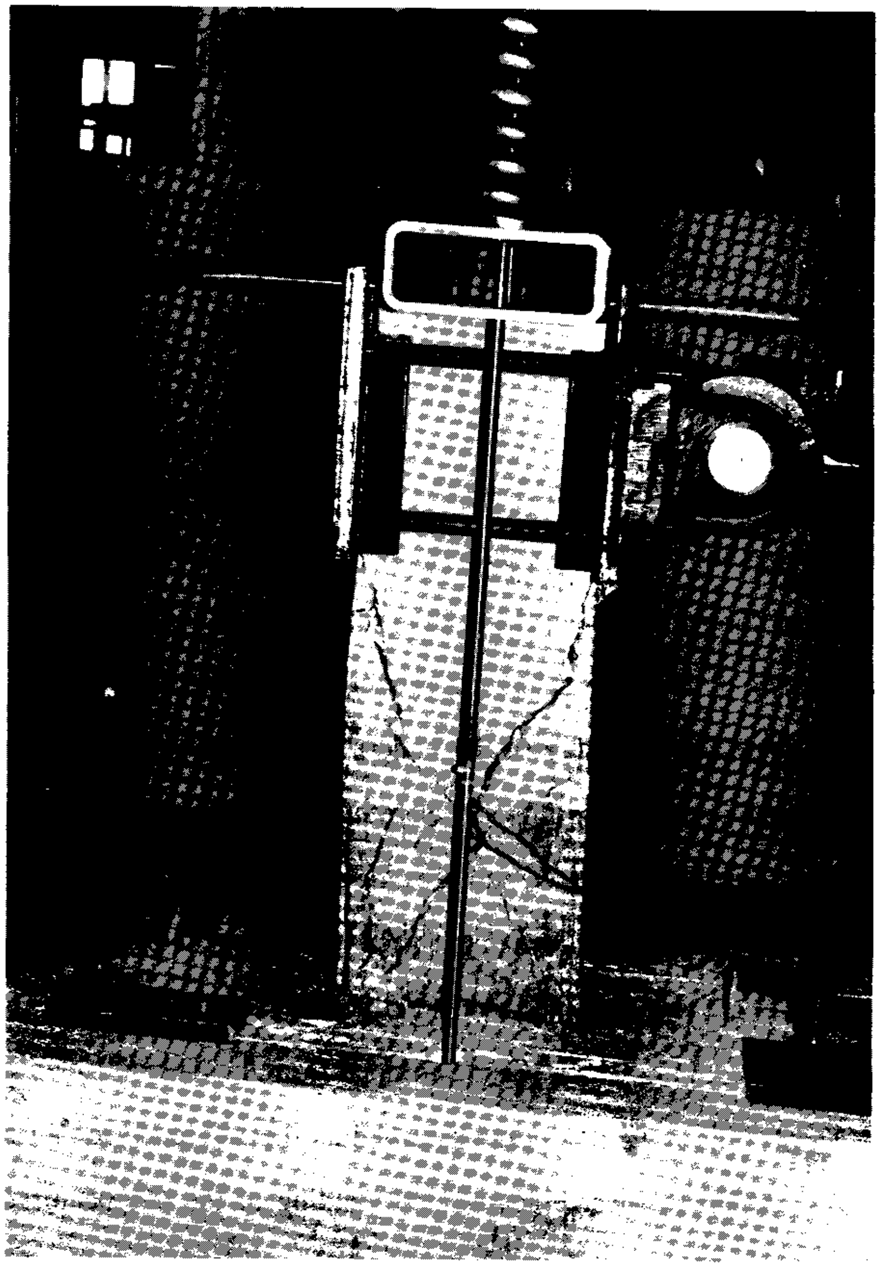

Figure 8. Shear Cracking in Control Column Specimen

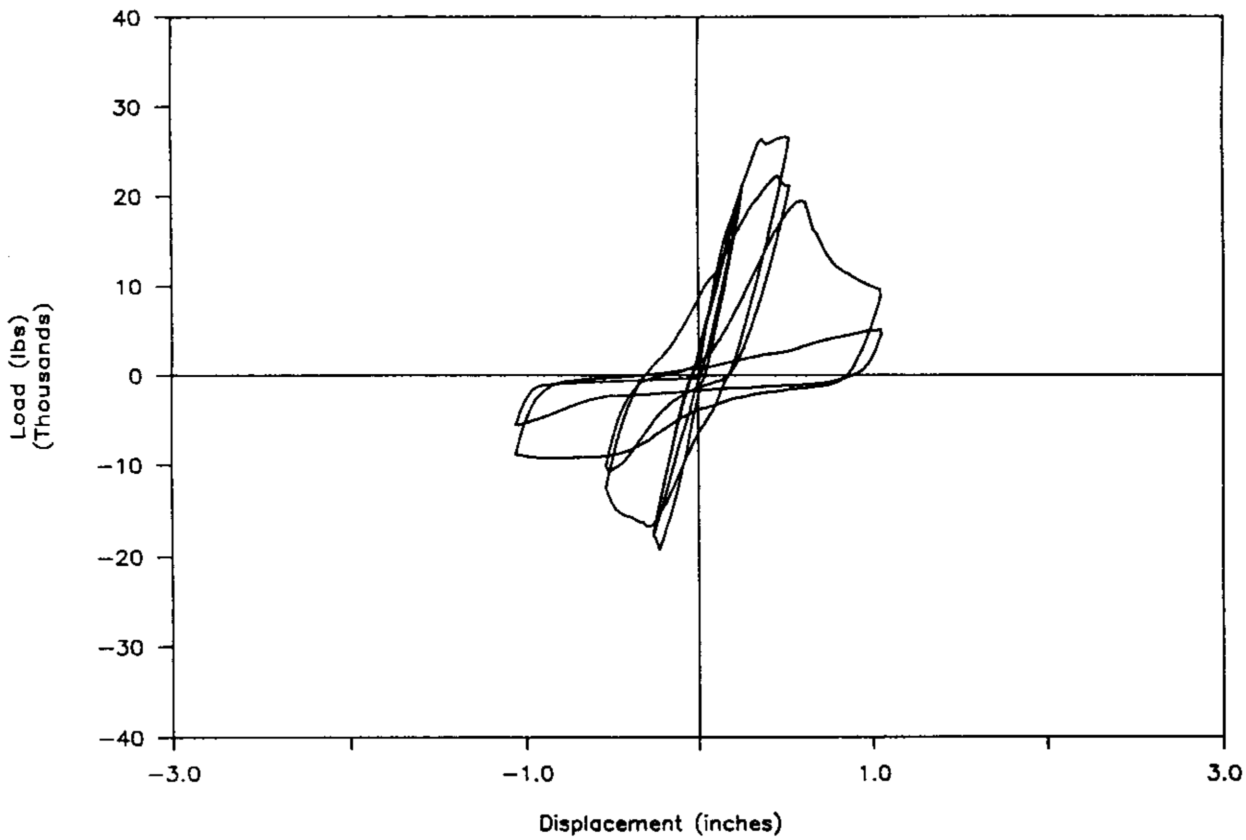

Figure 9. Lateral Load-Displacement Curves for Column No. 2

ANGLE AND ROD RETROFIT

Column Nos. 3 and 5 were tested with the angle and rod technique, and each had a different retrofit hoop spacing. For both columns, all bars in the retrofit hoops were uniformly prestressed to 50 percent of the bars' yield stress.

In column No. 3, the retrofit hoops were spaced at 6 in. on center. During testing, internal tie yielding was observed during the first cycle to 2 Δ_y. At this same displacement level, cracks developed between the corner angles at an inclination of approximately 45°. The load capacity dropped sharply in the first cycle to 4 Δ_y because of brittle fracturing in the threads of the external bars. The bar failure began at the bottom of the column and worked upward. Although the external rods were made of a mild steel (A36), this was not reflected in the mode of failure. When the retrofit hoops were lost, shear cracks similar to those observed in the control column progressively opened up. After these large shear cracks had formed, the load capacity continued to drop to almost zero. The lateral load capacity of column No. 3 increased 7 percent over that of the control column because of the addition of the retrofit hoops. A load-displacement hysteresis plot for column No. 3 is shown in Figure 10. This column was determined to have a ductility of $\mu = 2$.

In column No. 5, retrofit hoops were spaced at 4 in. on center. Before this test was conducted, the bars for the retrofit hoops had been annealed to produce a material that would respond in a more ductile manner than those used in column No. 3. Internal tie yielding did not occur in the testing of Column No. 5 until the first cycle to 4 Δ_y. Again, shear cracks formed between the corner angles at an inclination of approximately 45°. Column No. 5, with a smaller hoop spacing than in column No. 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, and bar failure began at the middle of the column and worked downward. When the retrofit hoops

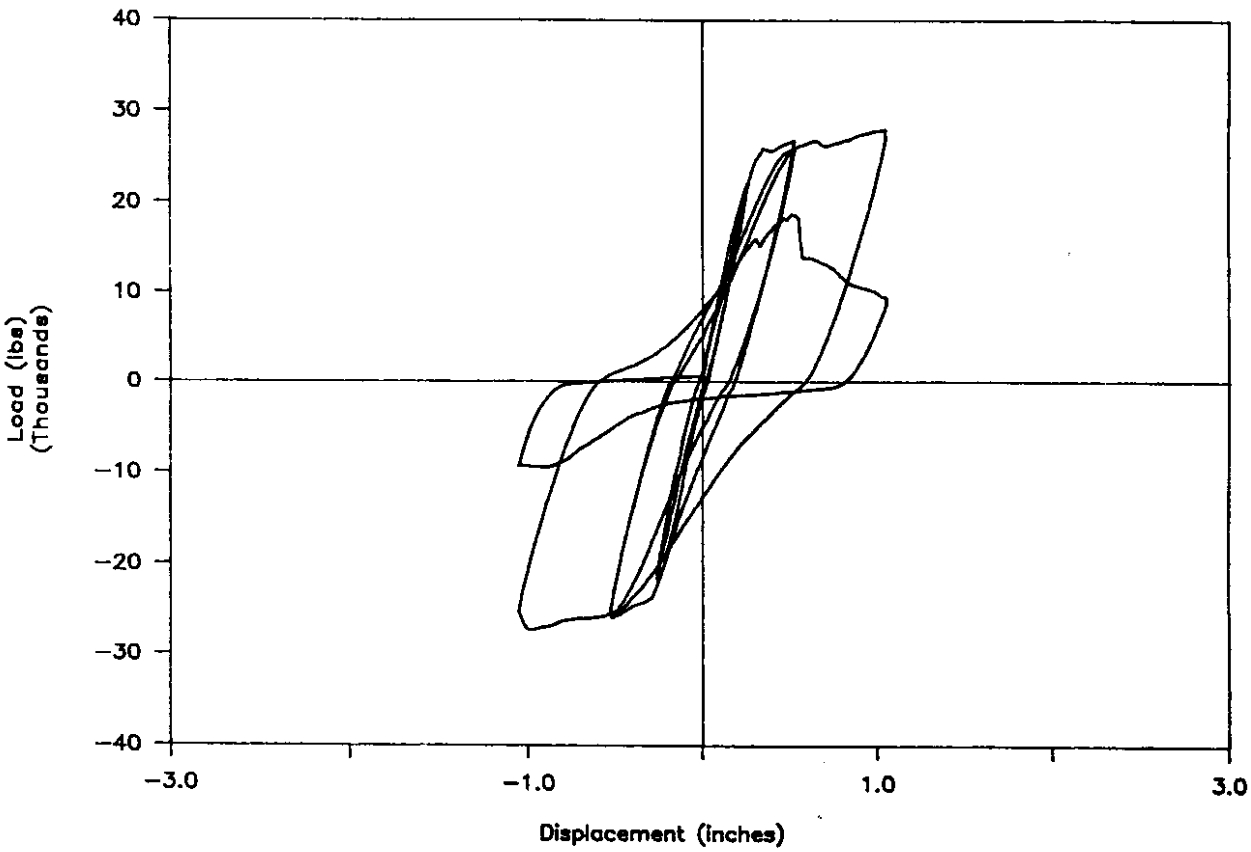

Figure 10. Lateral Load-Displacement Curves for Column No. 3

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, as shown in Figure 11. As with column No. 3, the lateral load capacity increased 7 percent over that of the control column. The ductility capacity of column No. 5 was greater than that of column No. 3, and the column's performance was good through $\mu = 4$. A load-displacement hysteresis plot for column No. 5 is shown in Figure 12.

This discussion shows that the strength and ductility of both columns with the angle and rod retrofit improved only moderately over those of the control column. Column No. 5, with the smaller hoop spacing, demonstrated a larger ductility capacity and slower internal tie yielding than column No. 3. Both columns showed an increase in lateral load capacity of 7 percent. Of note is the fact that shear cracks developed before the external hoops fractured, indicating that the mode of failure had not changed from shear to flexure.

The researchers had expected that an additional number of retrofit hoops would increase the shear capacity of the column enough that flexural failure would occur. However, the contributions to shear strength from the as-built specimen and the retrofit hoops were not directly additive. This discrepancy can possibly be explained by limitations in the ductility or elongation of the retrofit bars. Because of these limitations, the bars were not able to stretch sufficiently to accommodate the load without fracturing. The use of upset threads might improve this retrofit method's performance, but would significantly increase the cost of the retrofit.

STEEL JACKET RETROFIT

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

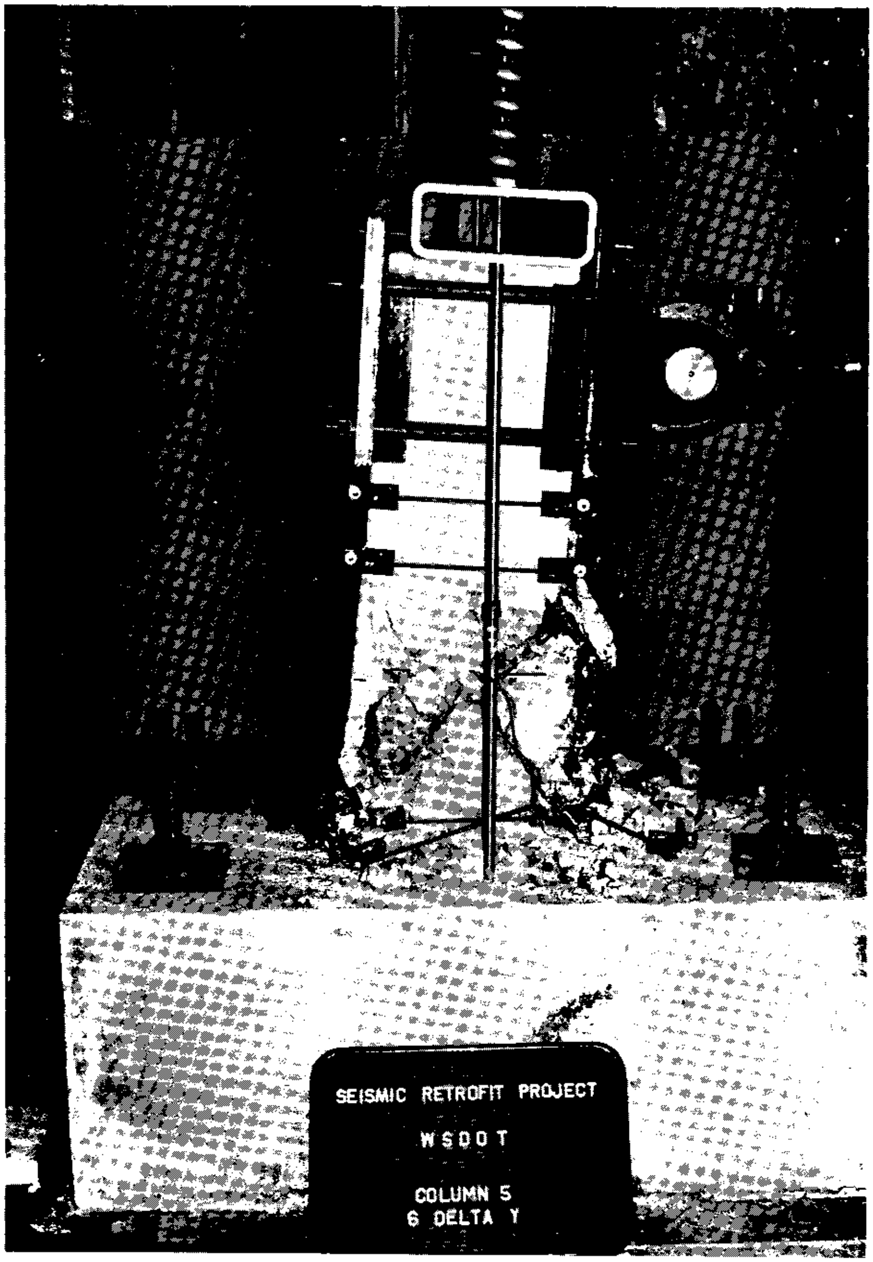

Figure 11. Column No. 5 After Testing

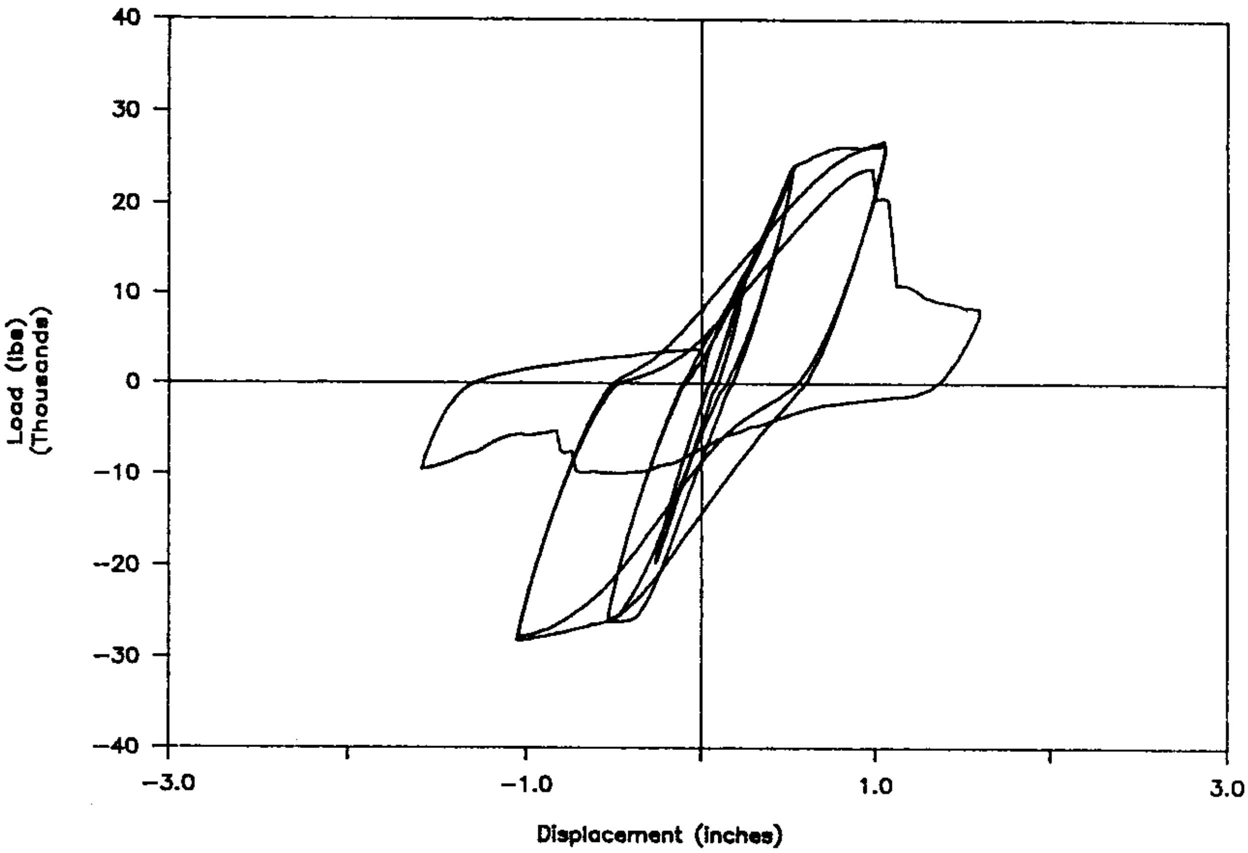

Figure 12. Lateral Load-Displacement Curves for Column No. 5

Column No. 4, which had a 16-gage steel jacket with non-shrink cement grout between the column and jacket, performed well under the imposed cyclic loading, with good load-carrying capability through a displacement ductility of $\mu = 8$. The column demonstrated a lateral load capacity enhancement of 16 percent over the unretrofitted specimen. The load-displacement hysteresis plot of column No. 4, displayed in Figure 13, shows that the retrofitted column dissipated energy well, and its performance was vastly better than that of the unretrofitted column. Beginning during the 4 Δ_y cycle, the column's longitudinal bars and steel jacket buckled near the base of the column at the maximum moment section, as shown in Figure 14. During testing to larger displacement levels, the 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 shear cracks was visible in the column.

Column No. 6, which was retrofitted with a 12-gage 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 No. 6 was 75 percent thicker than that in column No. 4, their lateral load capacities were the same. The load-displacement hysteresis plot of column No. 6, displayed in Figure 15, shows that the width of the loops was slightly wider than that in column No. 4, indicating good energy dissipation throughout the test sequence. As in column No. 4, the longitudinal bars and steel jacket buckled near the base of the column, beginning during the 4 Δ_y cycle. However, the extent of buckling was slightly reduced by the 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 by this

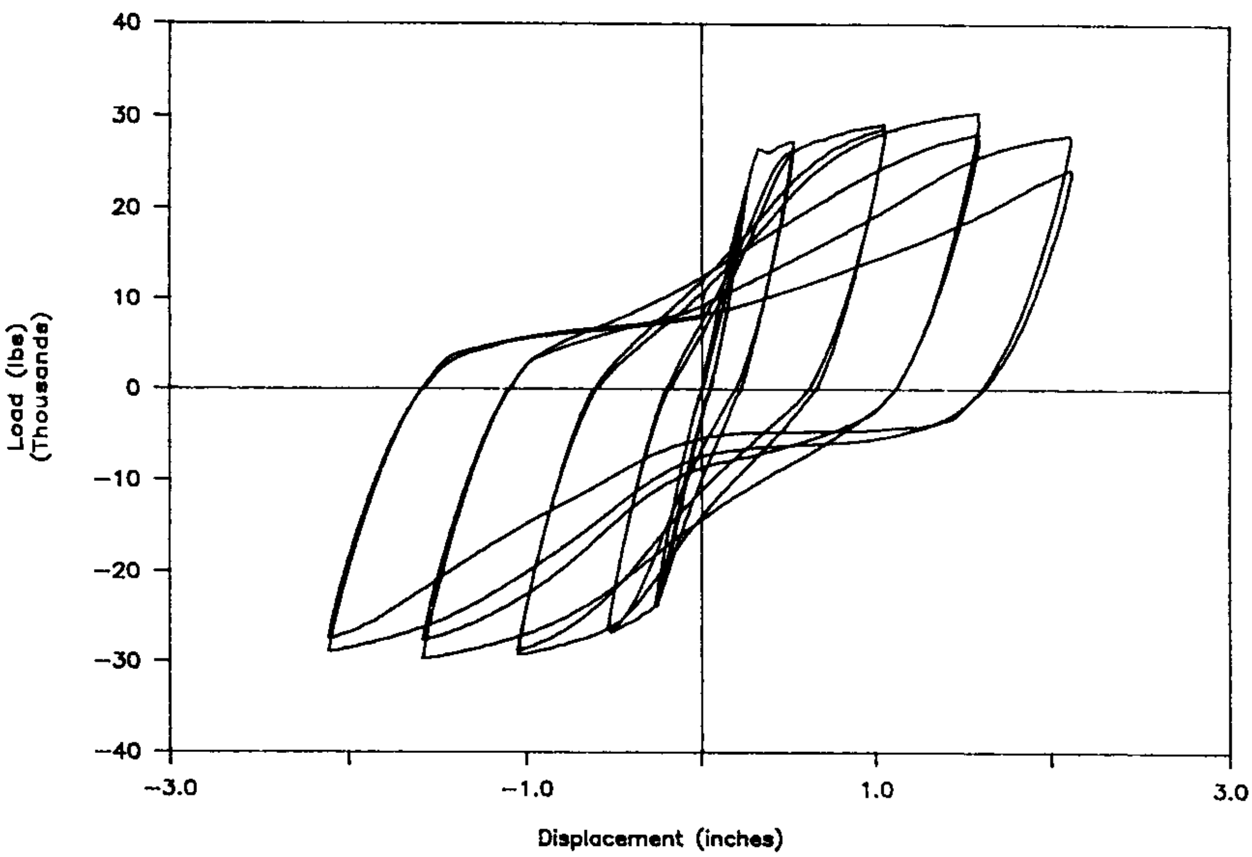

Figure 13. Lateral Load-Displacement Curves for Column No. 4

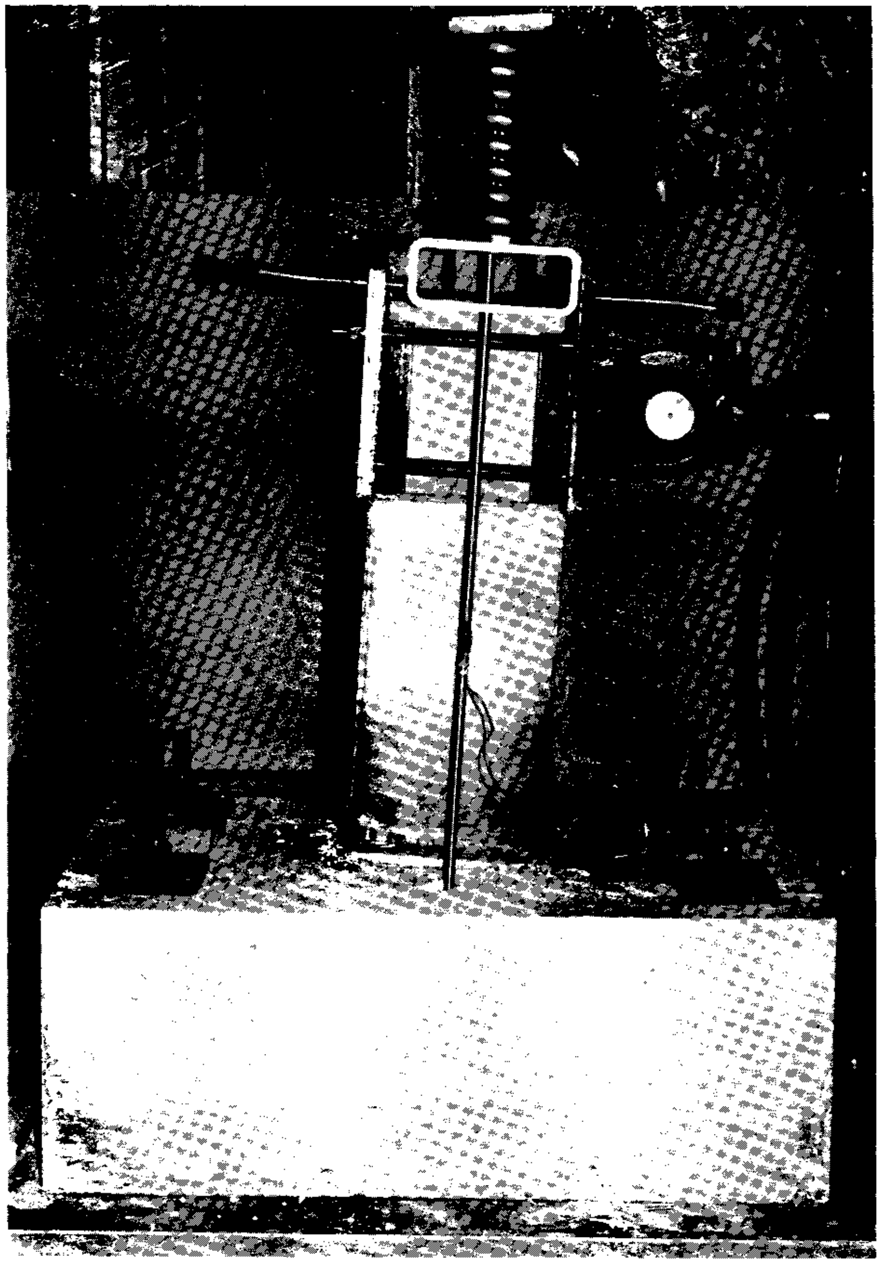

Figure 14. Buckling of the Steel Jacket in Column No. 4

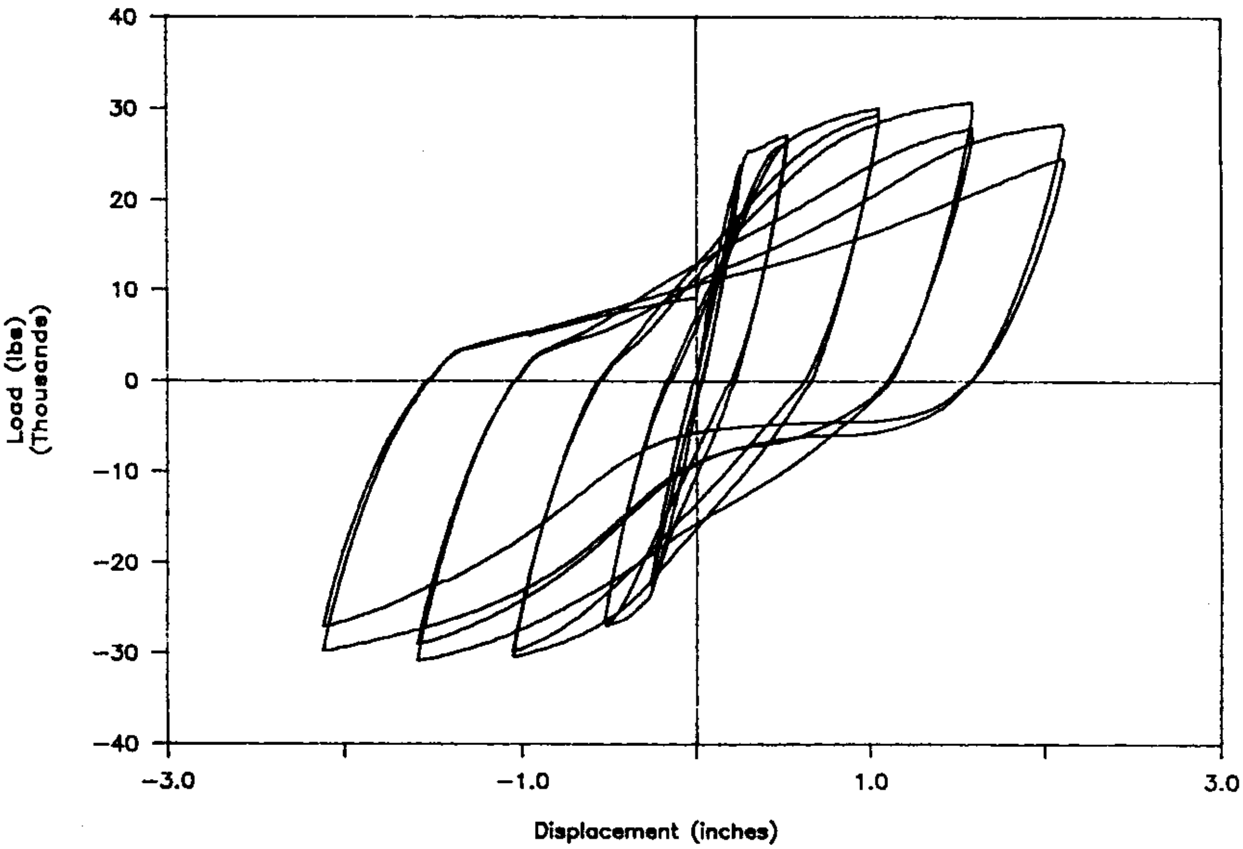

Figure 15. Lateral Load-Displacement Curves for Column No. 6

retrofit scheme, and no shear cracks were seen in the column after the jacket had been removed.

Column No. 7, which had a 16-gage steel jacket, cement grout, and four steel dowels in the column core, performed well through a ductility of $\mu = 8$. In this specimen, the lateral load capacity increased 19 percent over that of the control column. This was the largest increase of all the columns tested, possibly because of the increased confinement at the plastic hinge. In addition, the placement of dowels through the column core near the maximum moment section increased energy dissipation, as seen by the width of the loops in the load-displacement hysteresis plot in Figure 16. These hysteresis loops were the widest of all columns tested. The use of the dowels in the column core essentially eliminated buckling on one side of the column. However, buckling was seen on the other side of column No. 7, possibly because an internal tie was severed when the research team drilled into the core on this side of the column. As with the other jacketed columns, no shear cracks were seen after the steel jacket had been removed, and cracking in the footing was caused by the downward migration of the zone associated with plastic hinging.

Column No. 8, which had a 16-gage steel jacket with epoxy between the jacket and column, demonstrated good load-carrying capability through $\mu = 8$. This retrofit produced an increase in lateral load capacity 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 plot of column No. 8, shown in Figure 17, is almost identical to that of column No. 6 with the thicker jacket. The epoxy filler seemed to improve performance slightly over that of column No. 4, which used the cement grout, but not enough to justify the substantially higher cost of the epoxy. The use of epoxy between the column and jacket significantly delayed buckling of the longitudinal

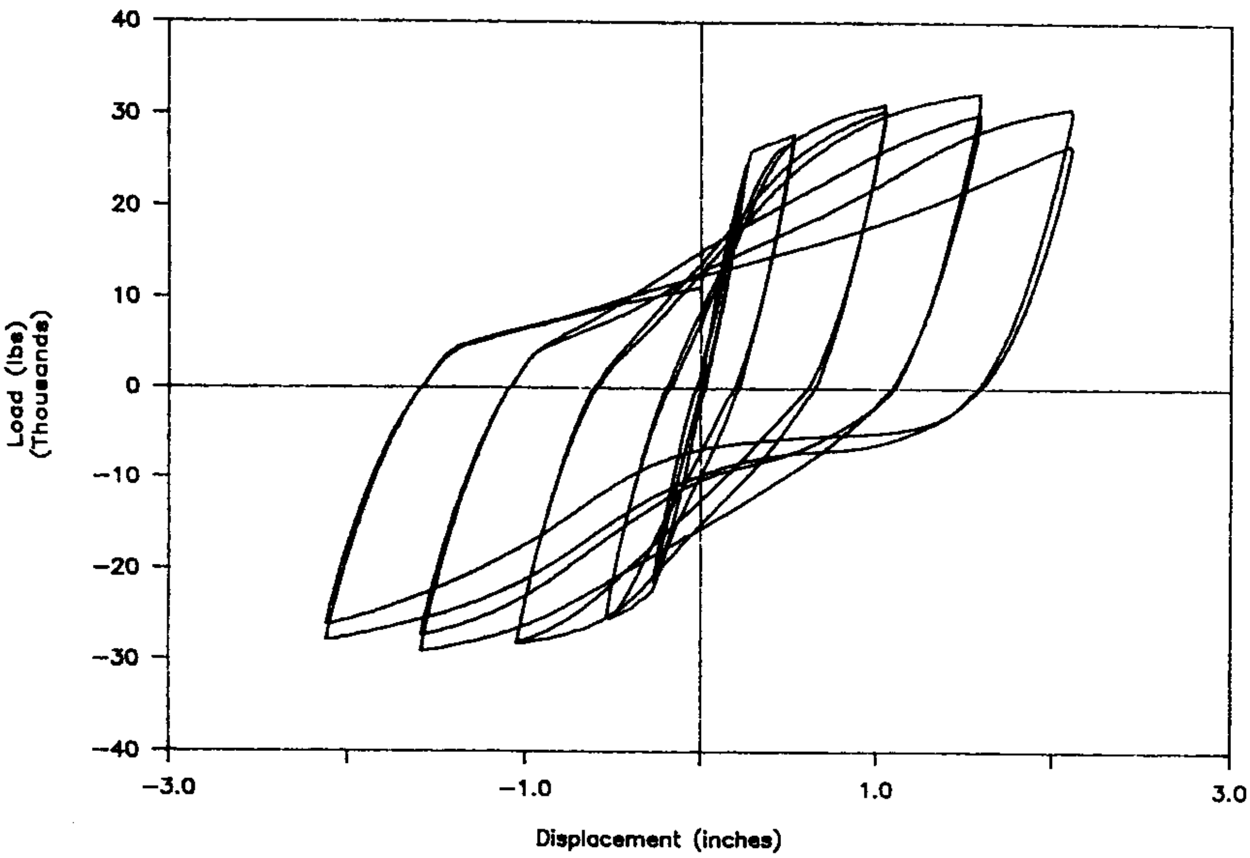

Figure 16. Lateral Load-Displacement Curves for Column No. 7

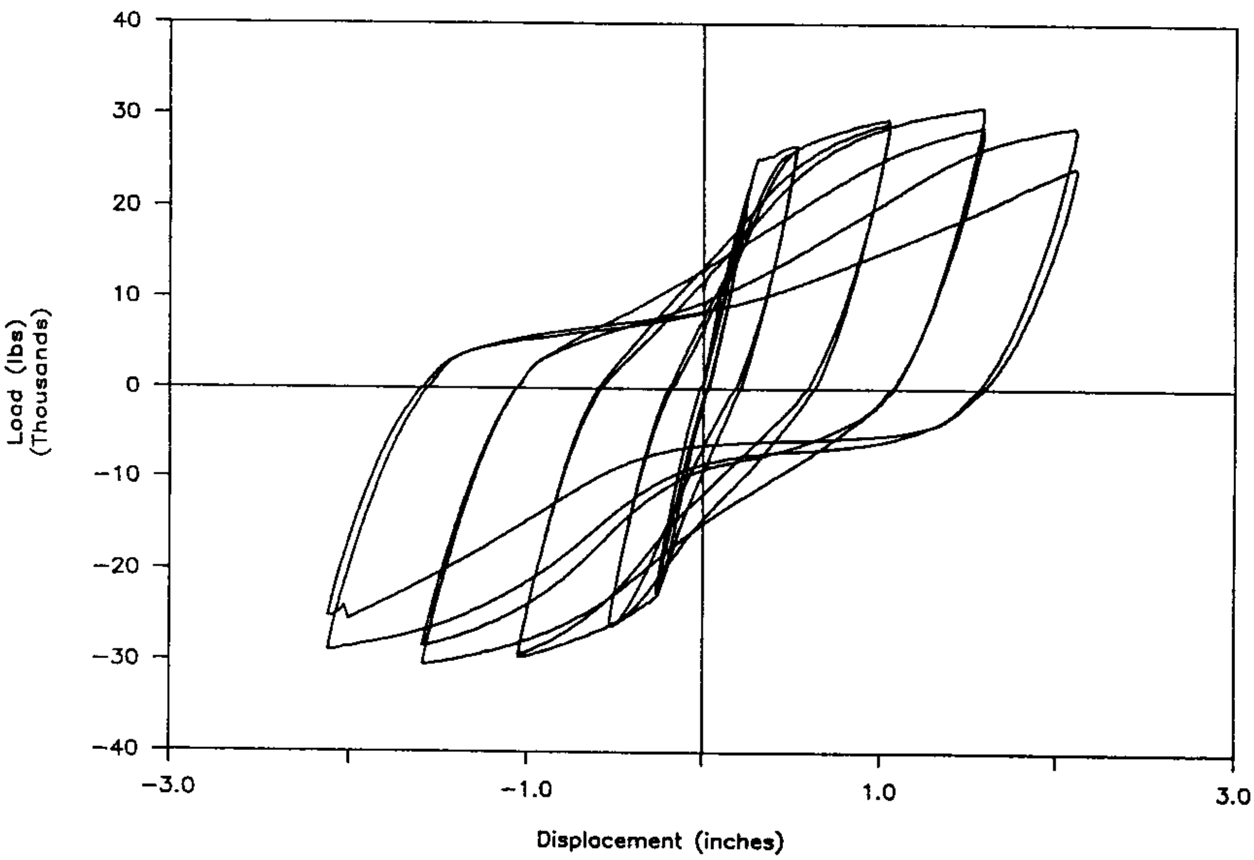

Figure 17. Lateral Load-Displacement Curves for Column No. 8

reinforcement and steel jacket, but buckling ultimately reached the same extent as in column No. 6. Cracking of the footing because of plastic hinge zone shifting also occurred.

This discussion shows that all the steel-jacketed specimens improved the strength and ductility capacity of the deficient section very well . In each column, a ductility of $\mu = 8$ was achieved with good load-carrying capability, after which testing was stopped. The load capacity increases were nearly uniform for all the jacketed specimens, ranging from 16 percent to 19 percent. The highest value came from the specimen that incorporated dowels in the column core. In all the jacketed columns, the initial stiffnesses were essentially the same, and the stiffnesses were only slightly larger than that of the control column. Internal tie yielding was prevented with 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 had changed from shear to flexure. The energy absorption characteristics of the jacketed specimens were improved substantially in comparison to those seen in column No. 2. The use of dowels through the column core at the maximum moment section produced the most energy dissipation of any of the specimens. Increasing the thickness of the steel jacket slightly improved the energy dissipation, as did using the epoxy mixture instead of a non-shrink grout between the column and steel jacket.

Throughout the testing sequence of each steel-jacketed specimen, the longitudinal reinforcement and jacket at the base of the column buckled under the imposed cyclic loading. The largest amount of buckling was seen in column No. 4, in which both sides buckled. Buckling was slightly reduced by a thicker steel plate. Use of the epoxy filler significantly delayed buckling, but it ultimately reached the same extent as that in other specimens. Dowels placed through the column core also reduced buckling, and on one side it was essentially eliminated. Although the jackets yielded because of buckling of the longitudinal bars, all the jacketed columns maintained some confinement in the hinging region. However, tests by Priestley, et al. (2) have shown that the 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.

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