Retrofitting of Reinforced Concrete Bridge Columns

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The seismic performances of one control and three retrofitted half-scale circular reinforced concrete columns were studied. The columns were 10 ft high, 18 in. in diameter, fixed at the base, and free at the top. An axial, vertical load of 158 kips was applied at the top, which was translated horizontally in a plane symmetrically about the vertical in such a way as to produce a maximum of four times the yield strain in the longitudinal reinforcing steel at the extremes of the first cycle. The columns were then oscillated between these extreme displacements of the first cycle until the lateral forces required to produce these translations approached zero. The measure of seismic durability used was the number of such cycles that a column sustained before losing structural integrity. The four columns were all made at the same time with 3,200-lb/in.^{2} concrete and the same reinforcing and ties. The arrangement was intended to model that of bridge columns of the 1960s. Three columns were retrofitted with prestressed, externally located circular ties at intervals along the lower 3 ft. The spacing and size of these ties varied from column to column. The control column sustained less than one cycle before losing structural integrity; the retrofitted columns sustained a minimum of 13 cycles.

Seismic design criteria for bridge columns have been changed considerably by the Applied Technology Council (ATC). Many bridges designed before the inception of the ATC-6 criteria are seismically deficient. Column inadequacies are usually evidenced by insufficient ties.

Inelastic response of these bridges under seismic loading could involve plastic hinging of the columns. Confinement of column reinforcing steel is essential for plastic hinging to dissipate energy under seismic loading without resulting in catastrophic column failure. Various details have been suggested for column retrofitting to increase confinement. Currently, there is no consensus as to which confinement detail is best.

The purpose of this research project was to determine the effectiveness of retrofitting circular concrete columns by adding external hoops made from standard Grade 60 steel reinforcing bars. To reduce parameters, columns were selected so that the predominant failure would be in flexure, which would satisfy two concerns: to fit the size and configuration of the existing loading frame, and to produce the predominant damage in the column itself. A cantilever column was selected so the retrofit hoops could be specifically located in the damaged region.

PROTOTYPE COLUMN DESIGN

AASHO first included earthquake design guidance in the 1958 interim (*1*). Column confinement was not officially incorporated for ductility purposes until 1983 when AASHTO published its *Guide Specifications for Seismic Design* (*2*). Soon after the 1971 San Fernando earthquake, bridge designers began to detail columns with additional confinement reinforcement. With the majority of Washington State's bridges built in the mid-1960s, during the interstate program, confinement reinforcement was not utilized for column ductility. A recent program by the Washington State Department of Transportation (WSDOT) (*3*) found 897 bridges in seismic WSDOT performance category C to be potentially vulnerable to a seismic event. Circular columns with deficient details exist in 65 percent of these bridges. The predominant details evolved from AASHO's specifications for highway bridges in the 1950s and 1960s. AASHO made three stipulations affecting the design of column confinement reinforcement. These three criteria are based on (a) compression reinforcing in flexure, (b) shear reinforcement, and (c) confinement for axial load. Buckling of flexural compression reinforcement required tie spacings not to exceed 16 longitudinal bar diameters. Shear reinforcement was required for concrete stresses exceeding an allowable of $0.03f'_c$ or a maximum of 90 lb/in.^{2}. Axial load requirements specified a minimum spiral volumetric ratio of

$$p_s = 0.45\ (A_g/A_c - 1)f'_c/f_{yh} \tag{1}$$

with a limiting size of a No. 2 spiral at 3-in. maximum spacing. Minimum tied confinement was more lenient with No. 3 hoops at 12-in. maximum spacing. These columns were allowed to carry only 80 percent of the axial load of the spirally reinforced columns.

WSDOT's typical circular column design consisted of a 3-ft-diameter column with 4-kip/in.^{2} concrete strength, a maximum aggregate size of 1.5 in., and 1 to 2 percent longitudinal reinforcement. All longitudinal reinforcement was lapspliced at the footing, with lengths ranging from 20 to 35 bar diameters. Most column designs were governed by flexure with an axial load range of 5 to 20 percent $A_g f'_c$. This concept resulted in the frequent utilization of No. 3 hoops at 12-in. spacing in lieu of the alternate No. 2 spiral at 3-in. spacing. These No. 3 hoops were constructed with a 2-ft 4-in. lapsplice without anchoring the splice tails into the column core. These columns were generally supported on footings that often had no top mat of reinforcement, leaving the footing vulnerable to failure. Although footing failure is a major concern, the main emphasis of this project is on the isolated capacity of the column. To determine the effectiveness of the column retrofit method, these details were incorporated into the modeled column.

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MODELED COLUMN DESIGN

The four modeled columns (Figures 1 and 2) are identified as Columns 1 through 4. Columns were 18-in. in diameter with 1.5 percent longitudinal reinforcement (nine No. 6 grade 40). The concrete mix was designed for a strength of 4 ksi at 28 days with a maximum aggregate size of ¾ in. Internal hoops were No. 3 grade 40 reinforcing bars spaced at 1 ft with a 1-ft 2-in. lapsplice without anchoring the splice tails into the column core. Dowel bars utilized at the base were lapspliced with 35 bar diameters to the longitudinal column reinforcement.

To match the bolt pattern of the existing test slab, the column footing was simulated by a steel base plate. The base plate was roughened by grinding gouges into the surface to simulate the construction joint. Footing dowels were threaded 2 in. into the base plate, welded with ⅜-in. fillet welds and normalized to relieve crystallization of the welded area.

Column 1 was the control specimen tested without additional external confinement. The full column elevation in Figure 1 shows Column 1 as tested. The partial elevations in Figure 1 combine with the control column to make up the three retrofitted columns. The external reinforcement was varied to determine its effectiveness. Ratios were selected above and below the requirements of AASHTO (*2*, p. 28).

External confinement reinforcement ratios are listed in Table 1. The percentage of volumetric ratio, p_s, is based on the gross section (*2*, p. 28) and is

$$p_s = 0.12\, f'_c / f_{yh} \qquad (2)$$

The percentages in Table 1 are based on $f'_c = 4{,}000$ lb/in.² and $f_{yh} = 60{,}000$ lb/in.².

Swaged couplers, designed to 150 percent of the specified yield strength of the Grade 60 ASTM A615 reinforcing bar, were utilized to attach the half circumferential external hoops. These hoops were prestressed to the column by swaging opposing threaded couplers to each end of the hoop. As shown in Figure 3, a machined stud was threaded into the couplers to pull the two half hoops tight. A tension test showed that tightening with an open-end wrench produced a stress in the hoops to an average of 50 kip/in.².

Retrofit hoops would require some type of protection. This could be accomplished by painting the hoops or placing an additional cover concrete by slip forming. Nominal reinforcement may be required for crack control. Neither of these schemes was included in this project.

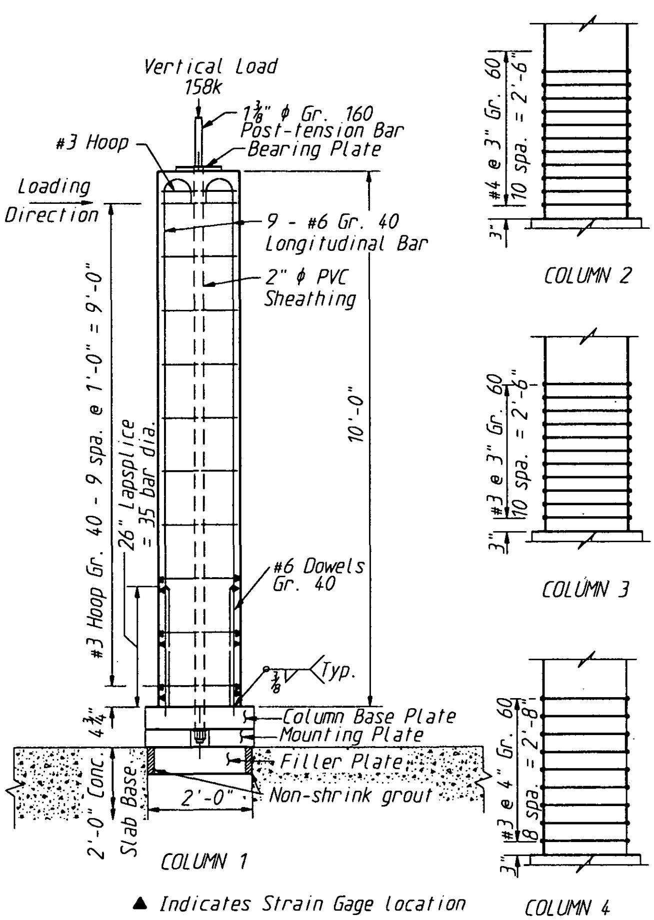


FIGURE 1 Column details and retrofit schemes.

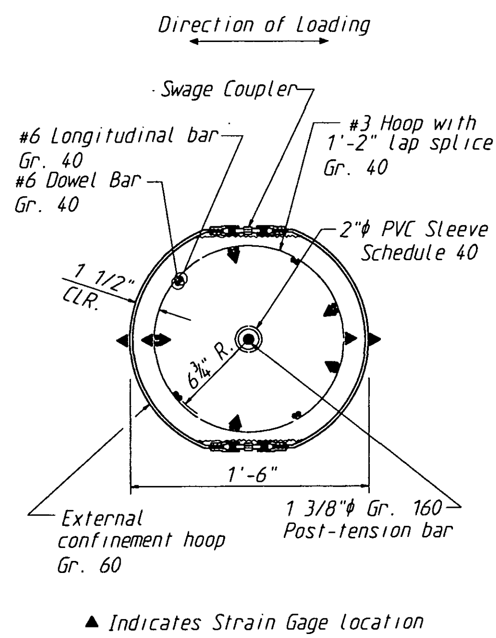
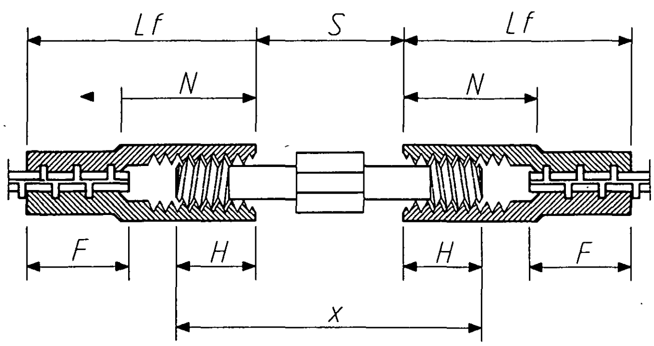


FIGURE 2 Column cross section.

TABLE 1 External Confinement Ratio

Column	Reinf. Ratio	Size/Spacing	% AASHTO
1	0.0	No Retrofit	
2	0.0152	#4 @ 3"	191%p_s
3	0.0083	#3 @ 3"	104%p_s
4	0.0062	#3 @ 4"	77%p_s

$p_s = 0.12 f'_c / f_{yh}$ (volumetric ratio)
All retrofit reinforcement ASTM A615 GR. 60



Dimension

Size	Lf	F	N	H	S	X	Weight coupler	Weight stud
#3	3"	1½"	1⅝"	9/16" min 1 3/16" max	⅝" min 1⅞" max	3"	.23 lb	.13 lb
#4	3¾"	1⅞"	1⅞"	11/16" min 1 15/16" max	⅝" min 1⅞" max	3¼"	.41 lb	.29 lb

FIGURE 3 Swaged coupler detail and dimensions.

COLUMN CONSTRUCTION

All columns were constructed simultaneously. Reinforcement was tied in cages and supported from the forms utilizing 1½ in. steel wire chairs. Concrete was placed in the vertical forms by concrete bucket and consolidated with probe vibrators. After curing for 10 days, the forms were removed and the columns were open-air cured (approximately 84 days) until tested. During curing, columns were freestanding with no axial load applied.

MATERIAL

The concrete mix design was based on WSDOT standard specifications for Class AX (4,000 lb/in.^{2}) concrete. Ready-mix concrete was purchased from a batch plant and delivered by truck. Concrete was designed as a four-sack mix with a ¾-in. maximum aggregate size. Slump was 3 in. at the time of delivery. Concrete properties were established by ASTM C469-87. Concrete test cylinders were taken throughout the placement of columns. Sample concrete was removed from cylinder forms after 10 days and air cured in situ with the columns. Average compressive strength was 3,200 lb/in.^{2} at 84 days and modulus of elasticity (E_c) was 3,176,000 lb/in.^{2}. The standard deviation of the concrete strength was 87.5 lb/in.^{2}.

ASTM A615 deformed bars were used for column reinforcement. Table 2 shows the size and average material properties. The modulus of elasticity was 29 million lb/in.^{2}. Couplers were swaged to reinforcement for tensile test. Coupler strengths equaled or exceeded 150 percent of the specified yield strength for the Grade 60 reinforcing bars. Table 3 lists a summary of the values from coupler test.

Strength and pretensioning of the couplers can be improved by using a higher-strength material for the stud. The extra strength will prevent the wrench from stripping the hex-head when tightened, allowing more tension to be exerted to the hoop.

TEST APPARATUS AND LOADING

Axial Loading

A total force of 158 kips was applied by a concentric rod and a centerhole hydraulic jack. The loading was monitored continuously by a load cell at the column top. The external hoops were placed before testing and before the axial load was ap-

TABLE 2 Reinforcement Properties

Size	Grade	Yield f_y (ksi)	Strength f_u (ksi)	Type of Reinforcement
#4	60	70.0*	94.0	Exterior Hoop
#3	60	81.0*	99.0	Exterior Hoop
#6	40	55.0	90.0	Longitudinal Reinf.
#3	40	63.0*	80.6	Interior Hoop

* Estimated yield. Actual yield was not determined due to prebending of reinforcement hoop bars.

TABLE 3 Coupler Data

COUPLER SIZE	INITIAL TIGHTENING (KIPS)	MAXIMUM TENSILE (KIPS)	FAILURE MODE
#4	9.35	20.0*	STUD
#4	10.5	20.7	BAR
#3	4.8	11.0	STUD
#3	4.7	11.0	STUD

* Ultimate specified force for a #4 is 18.0 kips, and #3 is 9.9 kips.

plied. The axial load produced hoop strains of approximately 4 percent of the total maximum strain during testing.

Horizontal Loading

The horizontal load was applied by a 110-kip MTS actuator. Each end of the load train was attached to MTS swivels. The swivels allowed rotation in both vertical and horizontal directions. The actuators were displacement controlled by computer analog converter at a maximum loading rate of approximately 0.4 in./min.

The actuator's total displacement stroke length was 10 in. Take-up in the load train limited cycling displacement to ±4.33 in. The first cycle had a maximum displacement of three-quarters of the yield displacement, followed by two cycles with maximum displacements of yield and twice yield, respectively. Loading continued with cycles of four times the yield displacement until forces required to maintain four times the yield displacement dropped significantly.

INSTRUMENTATION

Strain gauges were placed on dowels and longitudinal reinforcing bars at 3, 12¾, and 20¼ in. above the base plate on longitudinal reinforcing bars at five plan locations. Additional gauges were placed diametrically opposite on the bottom three internal hoops in the plane of loading. The external hoops were gauged in a manner similar to that for the internal hoops. Figures 1 and 2 show the strain gauge locations.

Lateral force was measured with a 110-kip MTS load cell placed integral with the load train. Lateral displacements were measured with a Temposonic displacement transducer. An isolated reference bridge was used to support the Temposonic. Absolute displacements were measured directly from the column at the same elevation as the load train.

DATA ACQUISITION

All data were recorded by an HP 3497A data acquisition/control unit. This unit interfaced with an HP 9216 computer that reduced data and stored values on 3½-in. HP 9121 disk drives. All voltage readings were normalized and zeroed. Data were collected under static conditions as the column completed each incremental step.

TEST RESULTS

For the purpose of testing, a yield displacement was defined as D_y = 1.07 in. (*4,5*). This is the displacement required to cause yielding of the extreme longitudinal reinforcement. The maximum ductility factor, $\mu = D/D_y$, for the test was 4.0.

Results of the testing are illustrated as hysteresis graphs in Figures 4 through 7 and as photographs in Figures 8 through 10. The hysteresis graphs are plotted as horizontal force versus horizontal displacement. A comparison of Figures 4 through 7 illustrates the significance of the retrofit method. Note that the initial cycles of the four hysteresis graphs show similar stiffness. This aspect is crucial to bridges because the retrofit method does not increase loading to the foundation.

The hysteresis curve for Column 1 (Figure 4) is unstable at 4μ. Evidence of a sudden drop in load-carrying capability is observed. The column failed to form a plastic hinge. Energy dissipation was poor in subsequent cycles.

The hysteresis curves for all three retrofitted columns (Figures 5 through 7) are very stable at 4μ. No evidence of sudden drop in load-carrying capability was observed, and the plastic

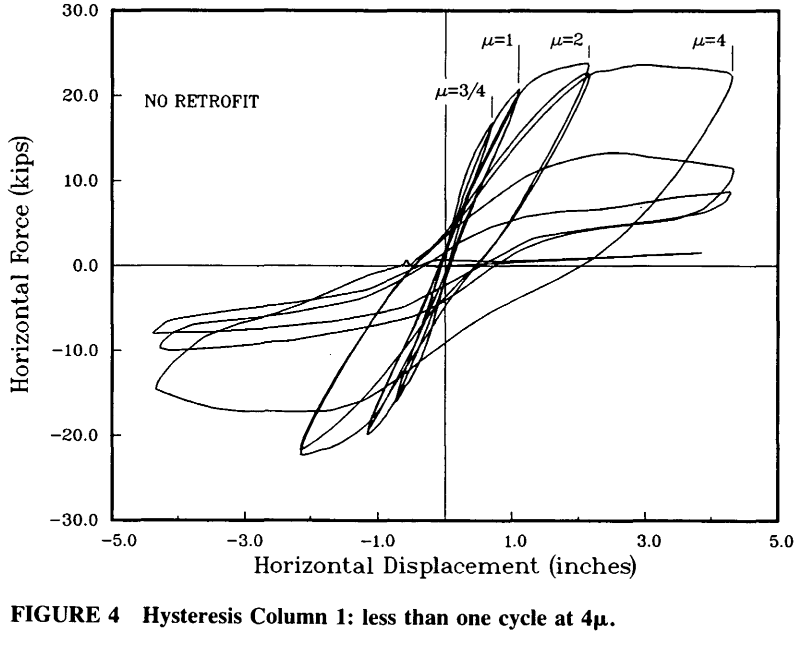


FIGURE 4 Hysteresis Column 1: less than one cycle at 4μ.

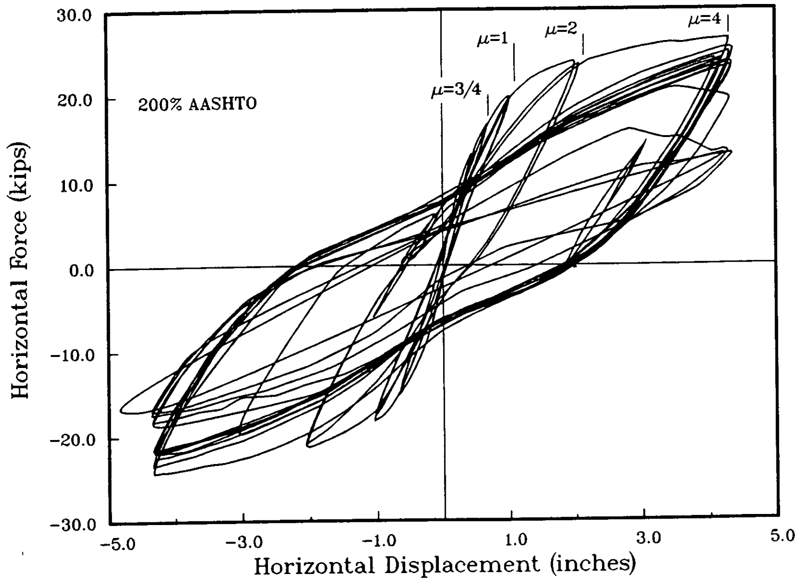


FIGURE 5 Hysteresis Column 2: 13 cycles at 4μ.

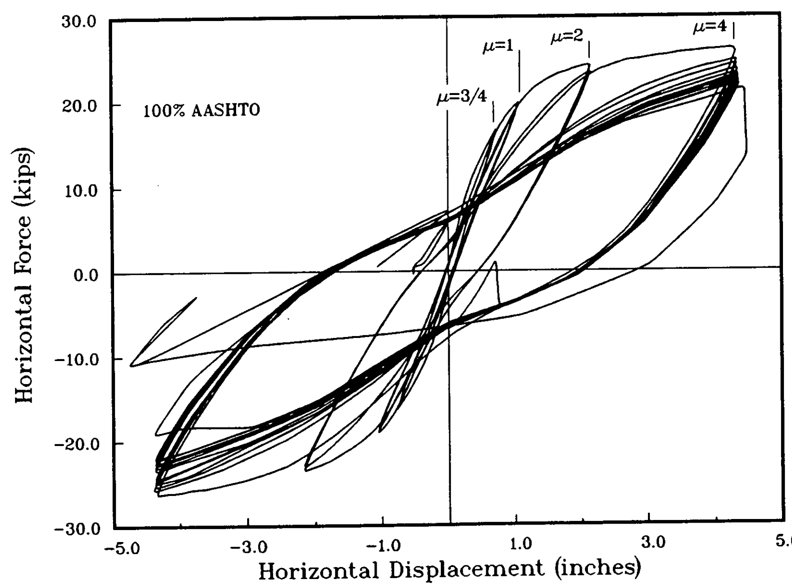


FIGURE 6 Hysteresis Column 3: 14 cycles at 4μ.

hinge continued to dissipate energy throughout the test. A large drop in load was evidenced near the end of testing when dowel reinforcement bars fractured at the weld to the base plate. The failure of the dowel at the weld is believed to be caused by embrittlement due to welding and was not a true characteristic of the prototype column.

FAILURE MODES

Two types of failures were encountered. The first type was in the lapsplice of the longitudinal reinforcement of Column 1. Figure 8 is a photograph of Column 1 at the completion of testing, illustrating the nature of the lapsplice failure. Notice that the concrete spalled over the entire length of the lapsplice. Failure began at 2μ with the development of a large flexure crack at the top of the lapsplice and concrete crushing at the base. The flexure crack increased in size, progressing into a diagonal shear crack as cycling proceeded to 4μ. A lateral translation kept the crack from closing, causing the concrete cover to spall, the lapsplice to slip, and the horizontal force required to attain 4μ to drop significantly. Completion of the second half-cycle resulted in the complete failure of the lapsplice. The hysteresis curve shows a force on subsequent cycles, which is attributed to the action of the posttensioning rod. In the final half-cyclē, the axial load was removed

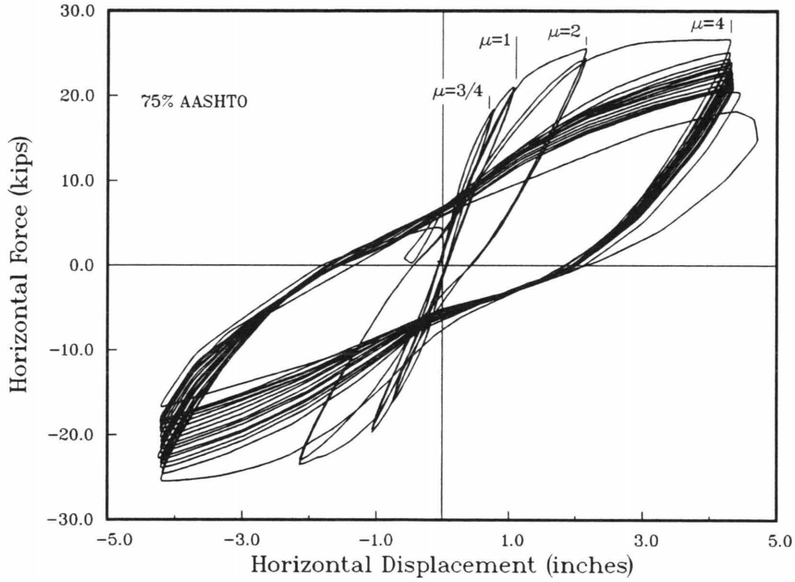


FIGURE 7 Hysteresis Column 4: 17 cycles at 4μ.

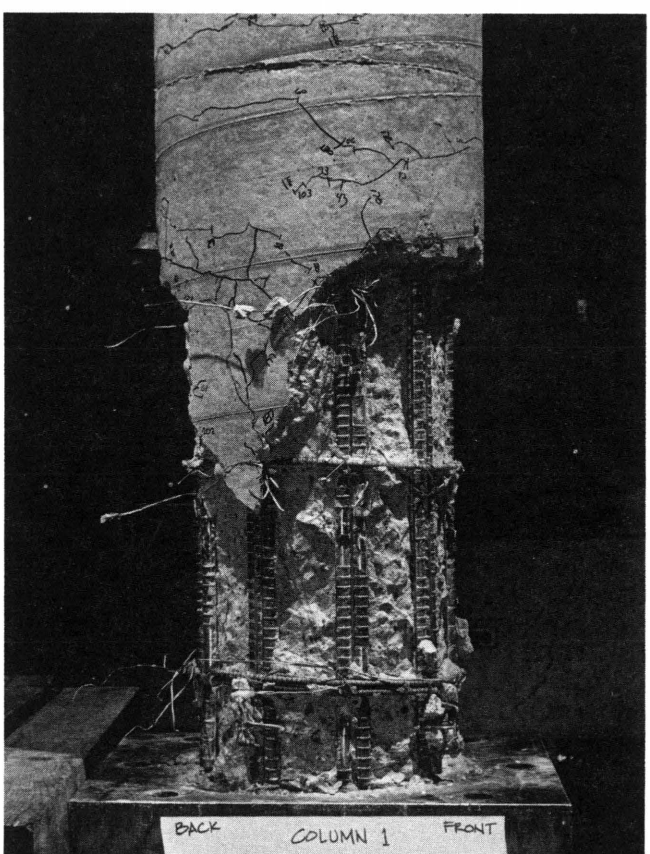


FIGURE 8 Column 1 elevation at completion of test: cover spalled but did not fall off and was easily removed manually for visual inspection.

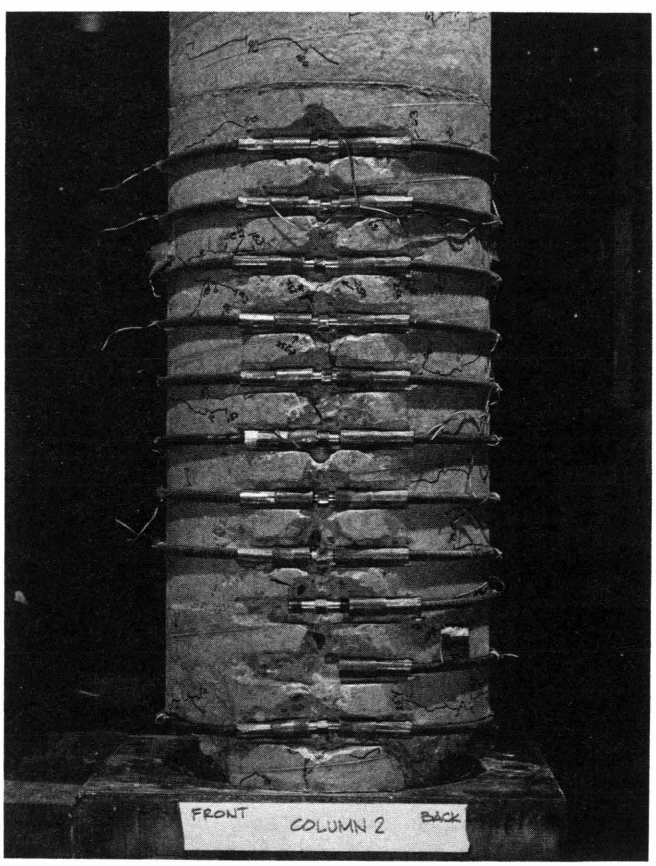


FIGURE 9 Column 2 elevation at completion of test: notice that damage is all located below bottom exterior hoop; hoops were undamaged; bottom hoops were removed at completion of testing.

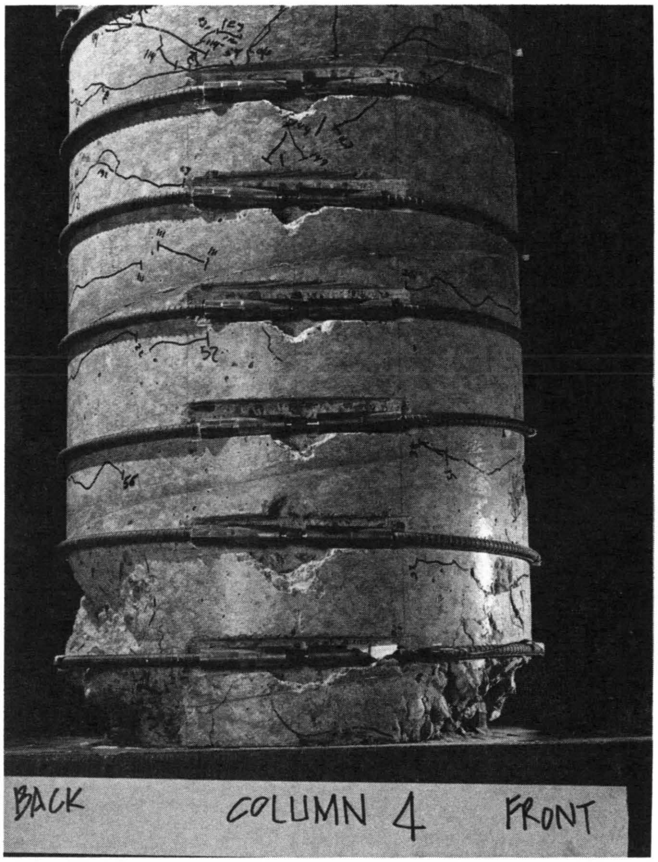


FIGURE 10 Column 4 elevation at completion of test: this column had minimum external confinement; concrete crushing developed above bottom hoop.

and Column 1 was displaced to 4μ. The horizontal force required to attain 4μ was 500 lb compared with the ultimate of 23,870 lb. The core concrete was unspalled at the end of testing and capable of carrying the axial load, although the section would be vulnerable to failure in shear or uplift with further loadings because the internal hoops were no longer effectively confining the lapsplice.

The second mode of failure was predominant in the three retrofitted columns and consisted of concrete crushing below the bottom external hoop. Figures 9 and 10 are photographs of Columns 2 and 4 after testing. These figures illustrate the second failure type. Notice that the majority of the damage is now located in the bottom 3 in. Concrete crushing at the base continued to propagate toward the column core until the dowel reinforcement fractured at the weld to the base plate. Dowel reinforcement fractured just above the fillet weld. Failure was restricted at the base because of the steel plate. A concrete foundation would allow crushing of concrete and a longer hinging length, which would be beneficial to performance under these testing conditions. Increasing the length of plastic hinging in turn increases the energy dissipation.

This method of retrofitting columns is successful in improving the column performance, but it does nothing for improvement of the footing. Potentially the failure will be located in the footing with the retrofit that has not increased the durability of the structure on the global scale. If it is impractical to retrofit the foundation, the alternative to do nothing at the column base may be plausible. The failure of Column 1 was essentially a hinge. If the column could then carry the loads at the top, no retrofit would be necessary at the bottom, which simplifies the entire retrofit to just the top of the column and cross beam.

Column 1 failed in slippage of the lapsplice after the concrete cover spalled. Spalling was largely caused by the wedging action of the deformations of the longitudinal reinforcement. Samples of longitudinal reinforcement had a small sheared wedge embedded on the compression side of the bar deformation. This wedging created a perimeter crack at the center of the longitudinal reinforcement where the concrete cover spalled away from the inner column core.

Columns 2 and 3 failed by concrete crushing due to flexural compression. Further damage was not evident until the displacement was increased. This pattern continued up to 4μ, where the dowel reinforcement fractured after a minimum of 13 cycles. Columns 2 and 3 retrofit hoops prevented the formation of a flexure crack at the top of the lapsplice. Column 4 failure varied from that of Columns 2 and 3, developing instead a flexural crack at the top of the lapsplice. The crack did not propagate into a significant size. Column 4 also had concrete crushing at the base propagate above the bottom external hoop during the initial cycles of 4μ. Figure 10 shows the damage above the bottom external hoop. No further damage above the bottom bar was observed in later cycles of 4μ. As testing continued, Column 4 settled into the same failure mode as the first two retrofitted columns. This damage is an indication that Column 4 was failing in the same manner as Column 1. Reflecting the retrofit reinforcement was close to the minimum required to prevent the lapsplice failure.

EXTERNAL HOOP REINFORCEMENT

The external confinement reinforcement showed no signs of distress. Strains in the bottom retrofit bars did reach 1.5 to 2 times yield strain. Strains in the retrofit hoops varied along the length of the main reinforcement lapsplice, decreasing as you move up the column vertically to mid-lapsplice and then increasing again to the top of the lapsplice. Retrofit bars above the lapsplice showed little variation in strain.

Figure 11 indicates strain profiles of the bottom internal hoops for columns 1, 2, and 4. Also shown in Figure 11 (*middle* and *bottom*) are strains of corresponding external hoops from Columns 2 and 4. The relative magnitudes show the force distribution between the internal and external confinement hoops.

Note the unstable nature of strain in the internal hoop of column 1. This instability follows the first displacement to 4μ when the cover concrete spalled. The interior hoop lapsplice begins to slip, causing chaotic strain readings. The internal hoop strains of the retrofitted columns indicate no evidence of instability because the external hoops carry a majority of the load. The strain in the interior hoops of the retrofitted column is a function of the exterior hoop confinement. Without the additional exterior hoops, interior hoops do not contribute to confinement and should be excluded in designing the confinement reinforcement.

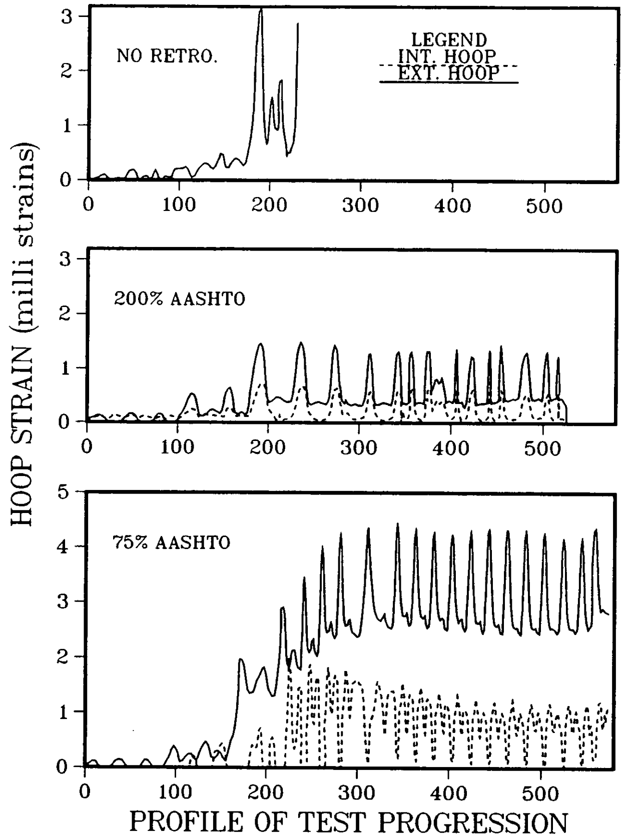


FIGURE 11 Strain profile of bottom hoops: *top*, Column 1; *middle*, Column 2; and *bottom*, Column 4.

INCREASED DUCTILITY

Ductility is a measure to withstand cyclic loading. Although it is not clear how much ductility a column should be designed for, ductility can be used to weigh the effectiveness of the retrofit method. The ductility of the retrofitted columns improved markedly over that of the control column. Energy dissipation was 269, 1,459, 1,565, and 1,842 kip-in. for Columns 1 through 4, respectively. Thus the energy dissipation as percentages of Column 1 are as follows: Column 2, 541 percent; Column 3, 581 percent; and Column 4, 684 percent.

Figure 12 relates the stability of the four columns as energy dissipation per cycle. Column 1 was unable to dissipate energy after the first cycle of 4μ. Retrofitted columns continued to dissipate a large proportion of energy after the first cycle of 4μ. Figure 12 also shows the duration of cycling at 4μ. The drop in Column 2 is caused by the continuation of testing after the dowel bars fractured at the weld.

The increase in energy dissipation with the reduction of confinement reinforcement can be attributed to the pinching of the longitudinal bar. As the confinement reinforcement decreases, the length to transfer force from longitudinal to dowel reinforcement increases. As the confinement reinforcement is decreased, strains increase when wedging separate cover and core concrete. This allows the longitudinal reinforcement to yield over a longer length, thus permitting an increase in strain energy of the longitudinal reinforcement and, thus, the amount of energy dissipated. With the larger confinement reinforcement, the strain energy of the longitudinal reinforcement is distributed over the bottom 3 in. of the column because of lower confinement reinforcement strains. This can be seen in Figure 9, where all the damage to Column

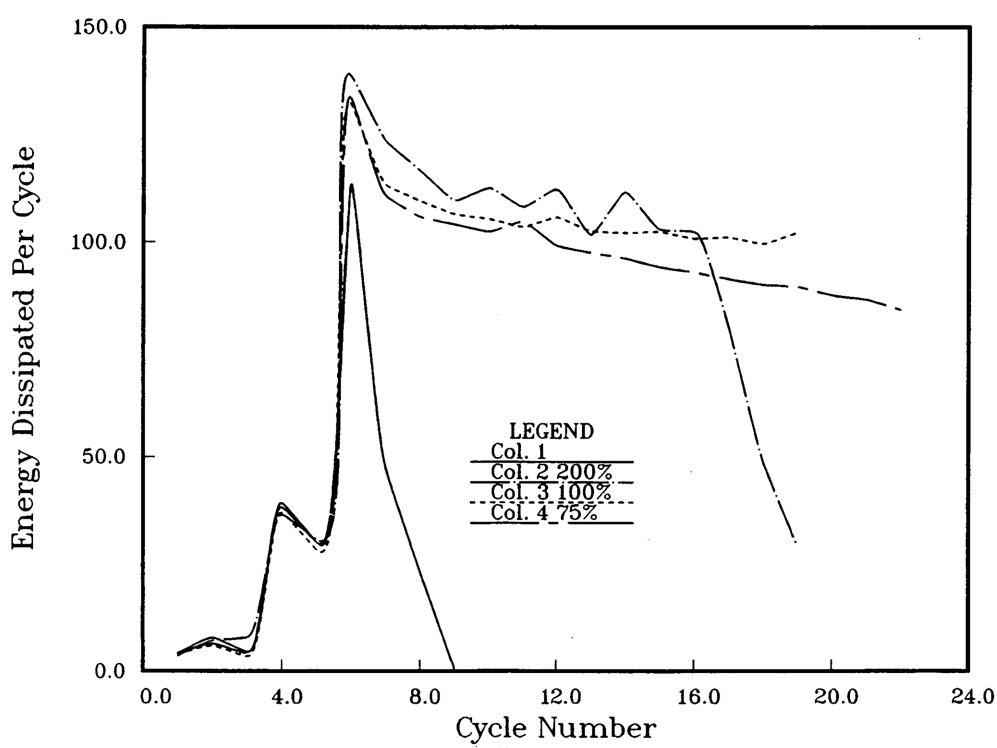


FIGURE 12 Energy dissipation per cycle.

2 is at the base plate. Column 4 in Figure 10 had damage above the bottom hoop, allowing about twice the length of longitudinal reinforcing to yield.

This increase in length was also evident in the strain profiles of Figure 11. Column 2 strains in Figure 11 (*middle*) are much smaller than those in Column 4 in Figure 11 (*bottom*). The No. 4 external hoops of Column 2 are large enough to confine the forces in the longitudinal reinforcement at the bottom hoop. It was apparent from the confinement hoop strains going up the column that the lapsplice was fully developed in half of its specified length; that is, the confinement hoops exhibited the greatest strains at the base of the column and just below the top of the lapsplice. The strain decreased toward the midpoint of the lapsplice.

SUMMARY

1. Installing hoops around the exterior as confinement improved the performance of the reinforced concrete columns by preventing lapsplice failure and thus increasing the ductility. Confinement is provided by exterior reinforcement, as indicated in Figure 11. Interior confinement had smaller strains than did exterior confinement and was unstable without exterior hoops such as those in Column 1 [Figure 11 (*top*)]. Design should exclude the contribution of internal hoops. Confinement should be provided as specified in the AASHTO guide specifications for plastic hinging (*2*). When retrofitting confinement is placed around the existing column, the gross area becomes equal to the core area, and the reinforcement ratio is limited by Equation 2.

2. Tensioning the exterior hoop is essential to performance. Confinement hoops should be tightened snug against the concrete. Although active confinement was not studied as a parameter in this project, previous studies (*6,7*) show that passive confinement reinforcement does not restrain the core until the column core fractures. This action generates a tension reaction in the confining reinforcement as the lateral strains increase markedly. Active confinement can increase strength by reducing core fracturing. The active confinement prevented the spalling of concrete cover, thus maintaining the lapsplice strength. Although no external hoops failed during column testing, hoop strengths were limited by the strength of the stud. Performance of hoops could be improved to provide additional strength and pretensioning by making the stud from a higher-strength material.

3. External hoops should be placed as close as possible to the supporting element to prevent concrete spalling. Column connections to footings and cross beams are regions of high stress during seismic loading. These connections are where the concrete is most likely to spall. To prevent large spalling in this locality, hoops should be as close as possible to the connecting part. Although shear failure was not investigated in this project, this retrofit method can be utilized in strengthening columns with insufficient shear capacity by standard design practice.

4. This retrofit technique is recommended for use at potential plastic hinge locations or in regions in which existing ties are insufficient to handle shear.

5. Foundation retrofitting is very difficult and expensive. Loads created in a seismic event are proportional to the column stiffness. Increasing loads to the foundation can generate expensive retrofits. To reduce costs, column retrofitting should thus minimize the effects to the foundation. The advantage of this retrofit is improved ductility without changing column stiffness. Although it is not clear whether it necessarily would be needed at the bottom of these columns, the retrofit could be used at the top. The reason to doubt the effectiveness at the bottom of the column is because of the need to retrofit the footing. It may be possible to eliminate footing retrofit by only retrofitting the top portion of the column.

ACKNOWLEDGMENTS

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