Seismic Durability of Retrofitted R.C. Columns

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16. ABSTRACT

The seismic performances of three retrofitted and one control, half scale, circular, reinforced concrete columns were studied. The columns were 10-ft. high, 18-in. diameter cantilevers. The longitudinal flexural steel was spliced to the foundation dowels just above the fixed base. A concentric axial load of .20 $f'_c A_g$ was continually applied during testing. The free ends of the cantilevers were translated to produce a maximum displacement of four times that necessary to produce yield in the longitudinal reinforcing steel. This loading was repeated with both positive and negative displacements in a quasi-static manner until the lateral forces required to produce these displacements approached zero. The measure of seismic durability used was the number of such cycles that a column sustained before losing structural integrity. The arrangement was intended to model that of bridge columns constructed during the 1960s. Three columns were retrofitted with prestressed, externally located circular hoops at intervals along the lower 4 ft. The spacing and size of these ties varied from column to column. The control column sustained less than two cycles before losing structural integrity; the retrofitted columns sustained a minimum of twelve cycles.

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Research Project GC 8286, Task 36
Bridge Column Retrofitting

SEISMIC DURABILITY OF RETROFITTED R.C. COLUMNS

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

INTRODUCTION

The metamorphism of bridge design and construction in regions prone to earthquakes has resulted in increasing structural safety. However, functioning structures that exist today were built to design standards of their period and, from a present-day viewpoint, their seismic resistances may have definite shortcomings. After every large earthquake, evidence of some of these shortcomings is revealed. The example of interest in this report is the absence of seismic durability in long, circular, prismatic reinforced concrete bridge columns built from 1950 into the 1970s. In these columns, the transverse ties provided were minimal and their effectiveness questionable.

Engineers accept that structural designs that result in a purely elastic response during large quakes are uneconomical. Therefore, they allow plastic hinges to form in regions of moment maxima during seismic lateral shaking. If these hinges are ductile, the structure's stiffness and integrity will be preserved. However, in the absence of the adequate confinement provided by transverse ties, the plastic hinge rotations, if large enough, result in concrete deterioration and radial movement of unconstrained longitudinal reinforcement. Since the 1971 San Fernando earthquake, the Washington State Department of Transportation, like other transportation agencies, has been studying and improving the seismic capacity of its existing bridges. Part of its studies has included the research reported here on the retrofitting of long, circular, reinforced concrete columns.

Figure 1 shows columns supporting an interstate highway that are typical of those constructed during the 1950s and 1960s. These columns were well designed for gravity loads and for the effects of wind, vehicle braking, and thermal changes. The return periods for these design loadings are small compared to the intended design life, and the structures have performed satisfactorily under conditions much like those specified in the design. However, in the case of earthquake loading, the return period is of the order of the design life. Therefore, it is unlikely that such a structure has experienced severe seismic shaking.

Figure 1. Typical Columns of the 1950-70 Period

The expected behavior can only be anticipated vicariously from that of similar structures that have been subjected to large seismic forces elsewhere. Thus, the behavior of bridges in earthquakes such as those in San Fernando in 1971 and in Loma Prieta in 1989 is significant in assessing the seismic capacity of structures in other parts of the country. Today, the columns similar to those illustrated would be designed so that plastic hinges would form at moment maxima in a manner that dissipated energy and yet maintained structural integrity, at least until the completion of the seismic event.

The lessons provided by these experiences have two effects on practice: the design requirements for structures are changed to reflect these new understandings, and existing structures are altered to ensure that their subsequent seismic behavior will be satisfactory. It is this second change, as applied to the retrofitting of long, circular, reinforced concrete columns, that was the concern of this study. The additions considered were external hoops wrapped and nominally tensioned in individual circles around the outsides of columns over the regions where local seismically induced moment maxima would occur. This form of cooperage was intended to confine both the cover and core concrete, as well as the longitudinal reinforcement, during an earthquake. To establish the effectiveness of this scheme, four identical columns, each 10 ft. long and 18 in. in diameter, were constructed of reinforced concrete from the same materials that have been used in the two decades of interest. Three of these columns were retrofitted in the manner described, with different combinations of external hoop size and spacing, and the fourth was unaltered. All were loaded axially to replicate the gravity effects and then cycled horizontally to simulate the seismic shaking. The lateral oscillations were quasi-static and continued until the columns lost their structural integrity. The number of cycles necessary to cause this degeneration was considered a measure of the seismic durability of the column. This retrofitting was found to provide a marked improvement in this measure of seismic competence.

This report begins with a discussion of the design of the experiments and then justifies the selection of the specimens in light of the design norms of the 1950s and 1960s and the anticipated retrofitting procedures of today. The methods of loading, constraining, and translating the columns are described, followed by a description of the data acquisition and reduction scheme. The results of the experiments and their analysis are presented, and, with the subsequent discussion, provide the basis for the conclusions.

RESEARCH APPROACH

DESIGN OF EXPERIMENTS

The retrofitting was intended to enhance the seismic performance of circular bridge columns built in the 1950s and 1960s in the Pacific Northwest. The experiments were intended to model these columns and the likely seismic experience in the area. Below, the contemporary design environment and the earthquake prognosis are first discussed, then the constraint of the available testing apparatus, and finally the experimental specification.

Design Environment

The columns designed in the 1950s and 1960s reflected the specifications of that period and any addenda of local officials. For instance, gravity loads of the 1961 AASHO specification (1) included dead, live and impact. The vertical live load was H20-S16, and 5 percent of the longitudinal lane load was included in the design. Lateral wind loads were augmented in 1958 for earthquake effects; these varied from 2 percent to 6 percent of the dead load, depending upon the foundation type and the site soil conditions. An example of local supplement is that the state of California had required consideration of these earthquake loadings since 1943. The columns of interest would have been designed for the same gravity loads and wind loads over the period of interest and may, or may not, have been designed for seismic lateral loads.

The design of columns at that time included ties for compression reinforcement, shear reinforcement, and core confinement. Respectively, buckling of compression reinforcement in flexural members was controlled by ties at spacing within 16 longitudinal bar diameters; shear reinforcement was required when shear stresses exceeded 0.03 f'_c or 90 psi; and minimum ties for column core confinement were 1/4-in. diameter hoops at 12-in. maximum spacing. The maximum concrete compressive strength, upon which allowable stresses were based, was limited to 4.5 ksi. The stress design method limited compression stresses to 0.4 f'_c in concrete and 13.2 ksi in structural grade steel reinforcement.

Typical columns designed to these specifications by the Washington State Department of Transportation had a 3-ft. diameter; a concrete strength of f'_c = 4 ksi; a maximum aggregate size of 1 1/2 in.; 1 percent to 2 percent longitudinal, grade 40 steel with lap splices at the footings varying from 20 to 35 longitudinal bar diameters; and #3 hoops at 12-in. spacing with 2-ft. 4-in. end laps with no anchorage into the concrete core.

Seismic Environment

Many of the columns of interest in Washington state exist in the seismically active Puget Sound and western Cascade regions. Noson and others (2) provided an overview of the region's earthquake hazard, from which the following synopsis has been drawn.

Earthquakes in this area are associated with the tectonic subduction of the small Juan de Fuca Plate beneath the much larger North American Plate. Two types of earthquakes are likely in the region: shallow ones with foci less than 30 km deep and below these, deep-focus events occurring at depths varying from 30 km beneath the coast to 100 km beneath the Cascades. Shallow earthquakes located within the North American Plate occur frequently and are often accompanied by aftershocks. Records over the last 150 years suggest a probable maximum magnitude of 6.5 for these events. Deep earthquakes are located within the subducting Juan de Fuca Plate, where larger events (magnitudes of greater than 4) tend to be more numerous than in the overlying plate. The largest recent events in the Puget Sound region were the 1949 Olympia (magnitude 7.1) and the 1965 Seattle-Tacoma (6.5) earthquakes. These two earthquakes, like smaller, recorded deep events, were not followed by aftershock activity. Their bracketed duration of strong shaking (based on 0.05g) was approximately 20 seconds (3) and 15 seconds (4), respectively. The maximum proposed magnitudes of these deep events has been set at 7.5.

A third type of earthquake has been hypothesized. This would be a large subduction earthquake with a magnitude exceeding 8 and would occur off the coast at a relatively shallow depth. The arguments for the occurrence of such an event are inferential and depend upon geological evidence.

Heaton and Hartzell (5, 6) on the basis of geophysical comparisons of the Cascadia subduction zone with other zones known to produce large subduction earthquakes, speculate that a single, giant, or series of large earthquakes could occur here with possible return periods in excess of 500 years. Other researchers (7) have found evidence of subsidence at several sites along the Washington coast. Large subduction earthquakes that have occurred elsewhere have produced residual, local elevation changes; thus, the evidence found here may support the subduction earthquake hypothesis. Finally, the lore of the indigenous people contains an account in a mid-January 1864 entry in the diary of the pioneer settler J.G. Swan that may be the description of either a tsunami or crustal deformation generated by a subduction event. (8, 9)

The first two types of earthquakes might be expected to produce similar intensities at the surface, since the larger events would be deeper.(10) However, the larger, deeper events would be felt over a much wider area. The hypothetical event would produce severe shaking and tsunamis along the coast, but attenuation would reduce its effects considerably around Puget Sound. The expected duration of strong shaking would be an order of magnitude longer than the more likely inland events could produce. (6) Local soft soil deposits could amplify the effects of all three types, and possible focusing of incoming seismic energy due to curved soil layers might also occur. (10)

Test Apparatus

The University of Washington, Department of Civil Engineering, utilizes an unusual arrangement to provide lateral loads to large model structures. This consists of a movable shear panel assembly, which is placed in a 2,400 kip Baldwin Universal Testing Machine (UTM). The resulting composite assembly provides a lateral load capacity of 100 kips at 10 ft. above an auxiliary reinforced concrete strong floor. The floor is prestressed in the direction of lateral load and is tied vertically to the main strong floor of the laboratory. Figure 2 shows this arrangement. The Baldwin UTM provides vertical forces and weighs 300 kips. This self-weight, along with that of the shear panels, is used to react the overturning couple produced by the lateral force on the shear panels. The shear panel assembly comprises two 12-in. thick, 6-ft. wide by 18-ft. high reinforced concrete panels, which are held 5 ft. apart by a steel trusswork. A heavy steel crossbeam is connected across the 12-in. edges of the panels. This beam allows the application of lateral loads at any point along the shear panel height and across a 10-ft. width. The scheme illustrated requires the UTM to apply a 1,000 kip vertical force to the shear panels. This force produces composite action between the UTM and the panels, thus allowing a maximum 100-kip lateral force to be applied to the assembly. A horizontal shear transfer strut connects the auxiliary strong floor and the shear panels to prevent sliding. Any specimen may be fixed to the auxiliary strong floor by 20-in. square steel base plates clamped by six 1 1/4-in. diameter, high strength bolts to embedded structural steel shearheads.

Experimental Specification

The purpose of this study was to examine changes in seismic durability associated with the retrofitting of existing bridge columns. A measure of durability had to be established. If structural integrity is preserved throughout a seismic event, then the retrofit can be viewed as a technical success. Local seismic features indicate moderately high intensities with possible long durations. This suggests a specification of the range of lateral translation. The number of repeated cycles at the specified range during which structural integrity is maintained becomes the measure of durability. In essence, we seek the increase in such durability produced by the retrofitting. The translation range is expressed as the ductility ratio—the ratio of the maximum translation to that translation needed to cause the first yield of the longitudinal reinforcement. Values of the ductility ratio, μ, of from 3 to 5 have been suggested for design purposes by Park. (12) Furthermore, Park cites the New Zealand Structural Design Code, suggesting that a structure should be able to withstand "at

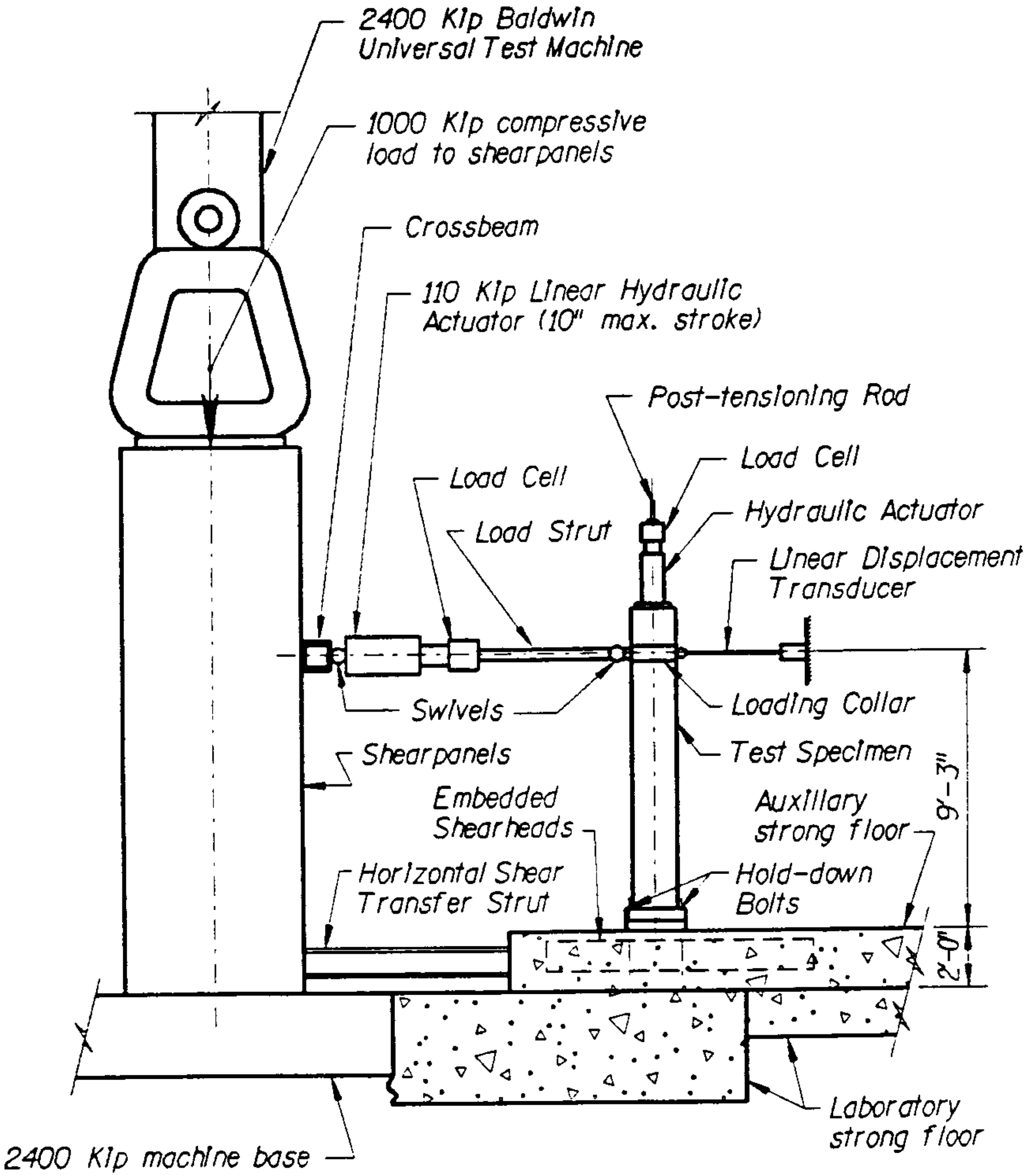

Figure 2. University of Washington Lateral Test Apparatus

least four times that at first yield, without the horizontal load carrying capacity of the building being reduced by more than 20 percent." This study adopted a level associated with the expected seismic experience in the Western Washington region, $\mu = 4$.

The test columns were to be reduced scale models of the prototypes, and yet the phenomenological behavior of the two had to be the same. Thus, the bond, shear, crushing, yielding, and stiffness characteristics of the materials were not to be modeled. This required that the test columns have sufficient dimensions for normal reinforced concrete behavior to be experienced and that the same materials specified for the prototype should be used in the models. The modeling scale was chosen as one half to ensure this normal behavior. This choice led to a minimum column diameter of about 18 inches.

The spacing and cover of the ties are critical measures. The spacing at 12 in. in the AASHO specification (1) is independent of the external column or core dimensions. The dimensionless ratio of tie spacing to cover is a possible variant in relation to spalling and confinement. The retrofitting proposed involved removing a part of the cover. These considerations indicated the importance of maintaining the tie spacing and cover of the prototype in the model.

The purpose of the experiments was to exhibit a change in durability when bending the columns. Therefore, the test members had to be long enough, in relation to the lateral dimensions, for the dominant behavior and failure modes to be flexural and not shear. In the case of full rotational restraint at both ends, this could be accomplished with a 20-ft. model length. The tests were to be conducted on a cantilevered member and so required a length of 10 ft. This dimension was within the limits of the test apparatus.

The columns would experience seismic motions while simultaneously experiencing at least the axial forces caused by the sustained deadload. Typical Washington state bridge designs of the period of interest resulted in column axial load ranges of 5 to 20 percent $A_g f'_c$. The higher levels produce the most brittle behavior, thus .20 f'_c was the required average concrete compressive stress due to deadloads

The confining devices which would be used to retrofit the test columns would have to be similar in their action to those envisaged for use on prototypes.

TEST ARRANGEMENT

The control column is described first, followed by a description of the retrofit method. The construction and material control procedures are then described, followed by an account of the measuring instruments, data acquisition, and loading systems.

Control Column (Column 1)

Figure 3 shows the 18-in. diameter test column — a half-scale model of a typical bridge column of the 1950s and 1960s. The gravity load was applied with a hydraulic ram at the top of the column. The ram reacted against a 1 3/8-in. diameter, high strength rod, which passed through a 2-in. diameter hole in the center of the column and was anchored against the column baseplate. The concentric, simulated, gravity load of 158 kips produced the same deadload stresses as in a typical prototype column. The reaction of the column was through the auxiliary floor slab previously described, where the lap splice dowels were screwed and welded to the 5-in. thick column base. Nine #6 dowels were spliced with #6 longitudinal bars ventered on a 6 3/4-in. radius. These splices were 26 in. long and the maximum length (35 diameters of the longitudinal steel) of that time. The longitudinal bars and dowels were surrounded by #3 hoops at 12-in. centers, with 1-ft. 2-in. lap splices, which were not extended into the core. The resulting cover to the #3 hoops was 1 1/2 in.

The materials were grade 40 reinforcing steel and 4-ksi concrete (28-day strength). The 5-in. thick steel column base was roughened by gouges in the surface to provide enhanced shear transfer capacity.

A steel collar was mounted around the column at the top and connected to the lateral load device. Figure 4 shows a photograph of these arrangements.

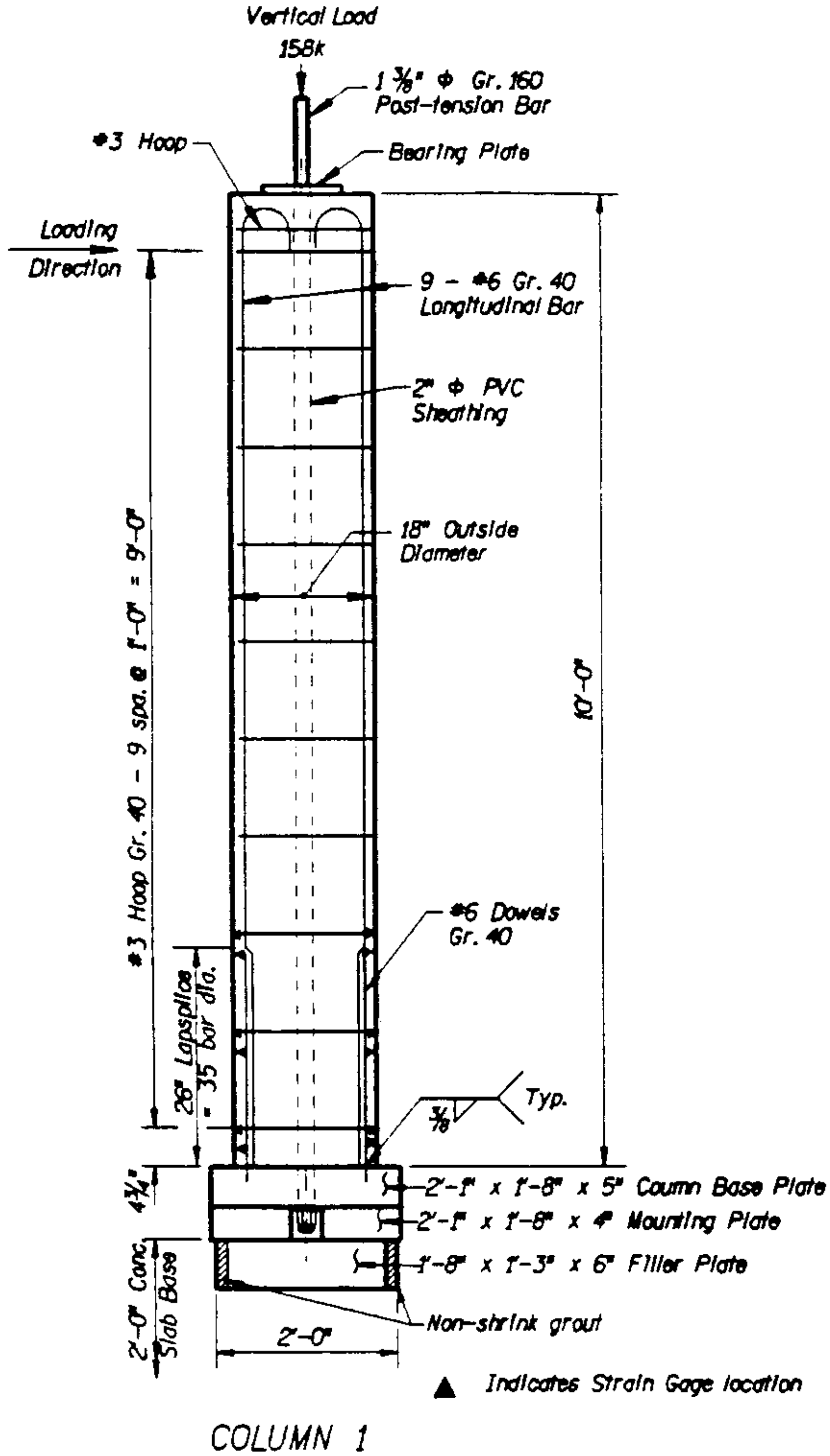

Figure 3. Control Column — Column 1

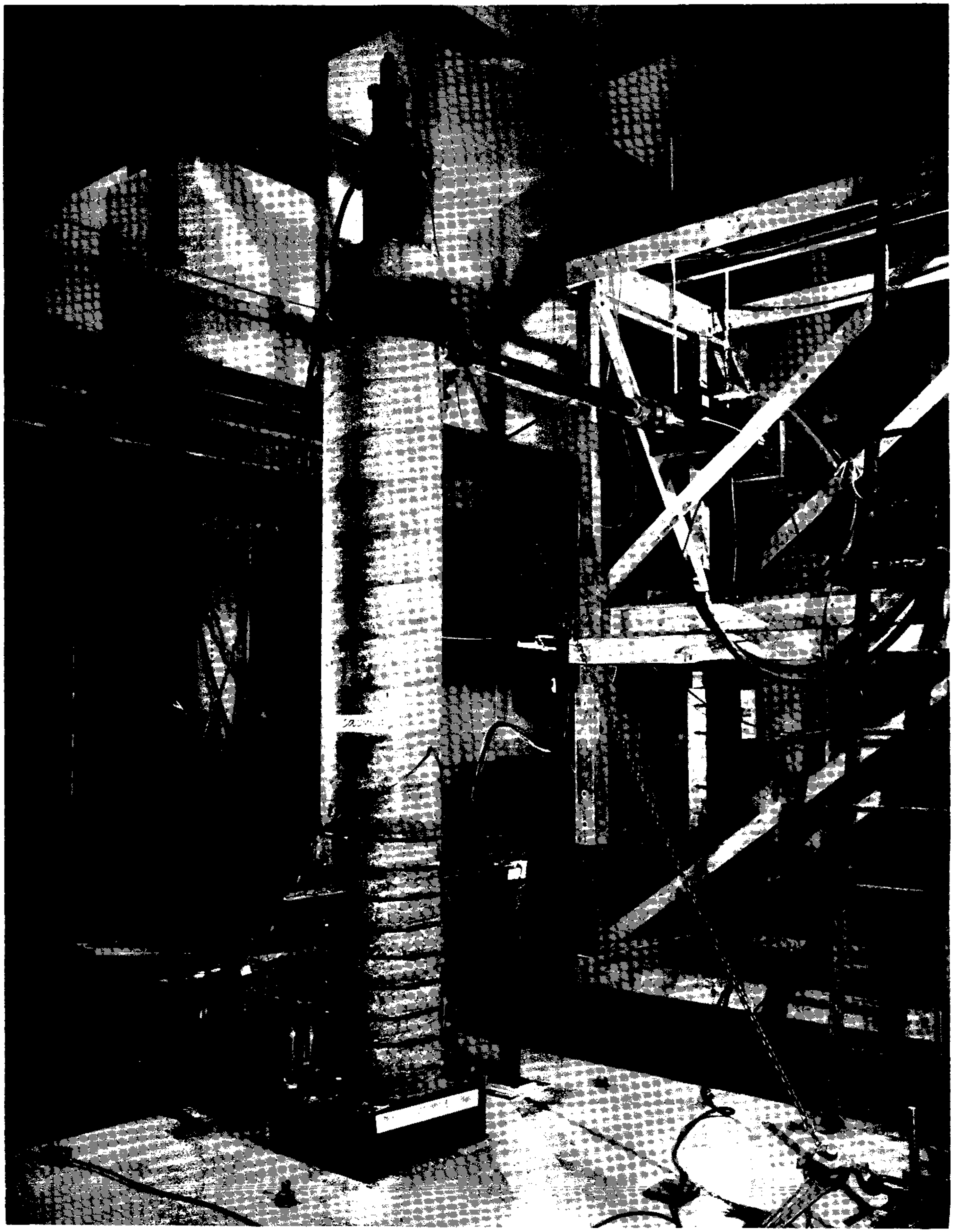

Figure 4. View of Test System

Retrofitting (Columns 2, 3 and 4)

Radial constraint was provided only over the length of the column where the longitudinal bars were spliced with the dowel bars. This length also included the potential zone of plastic hinging. The constraint was to be provided by discrete, prestressed, external hoops. Figure 5 shows the modifications to the control column, Figure 6, a cross-section of the retrofit, and Figure 7, the details of the swage coupler.

The spacing and size of the retrofit hoops varied from column to column. A measure of this is the confinement ratio, ρ_s, the volume of the confinement steel to the associated volume of column confined. In this ratio, the 18-in. diameter of the column is used to determine the concrete volume. Table 1 indicates these values.

The retrofit hoops were grade 60 ASTM A615 reinforcing bars. Figure 7 shows the way in which the hoops were connected by swaging opposing threaded couplers to the hoop ends. By tightening the machined stud of these couplers with a wrench (Figure 7), prestress was induced into the hoops. The strains in the hoop were measured during tightening; hoop prestress levels of approximately 50 ksi were obtained.

Construction and Materials

The four columns were constructed simultaneously. The reinforcement was caged and supported in the vertical forms; the concrete was placed by concrete bucket and compacted by probe vibrator. The forms were removed after 10 days, and the concrete was subsequently air cured until testing. The total cure time exceeded 80 days. While

Table 1. Retrofit Information

Column	Retrofit	ρ_s
1	0	0
2	11 #4	0.0152
3	11 #3	0.0083
4	9 #3	0.0062

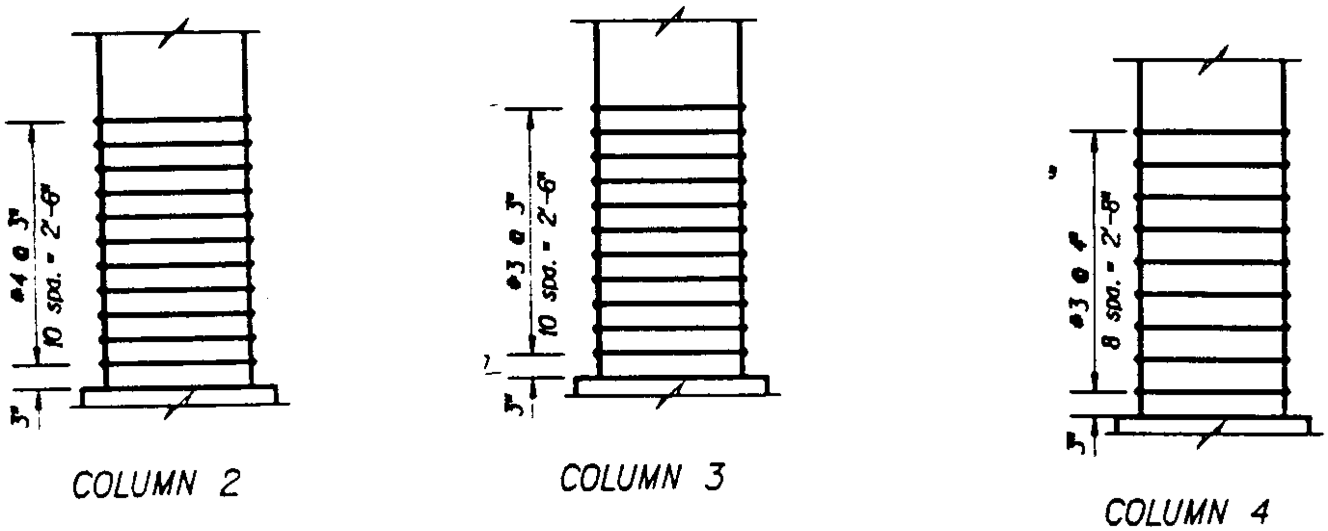

Figure 5. Retrofitted Columns — Columns 2, 3 and 4

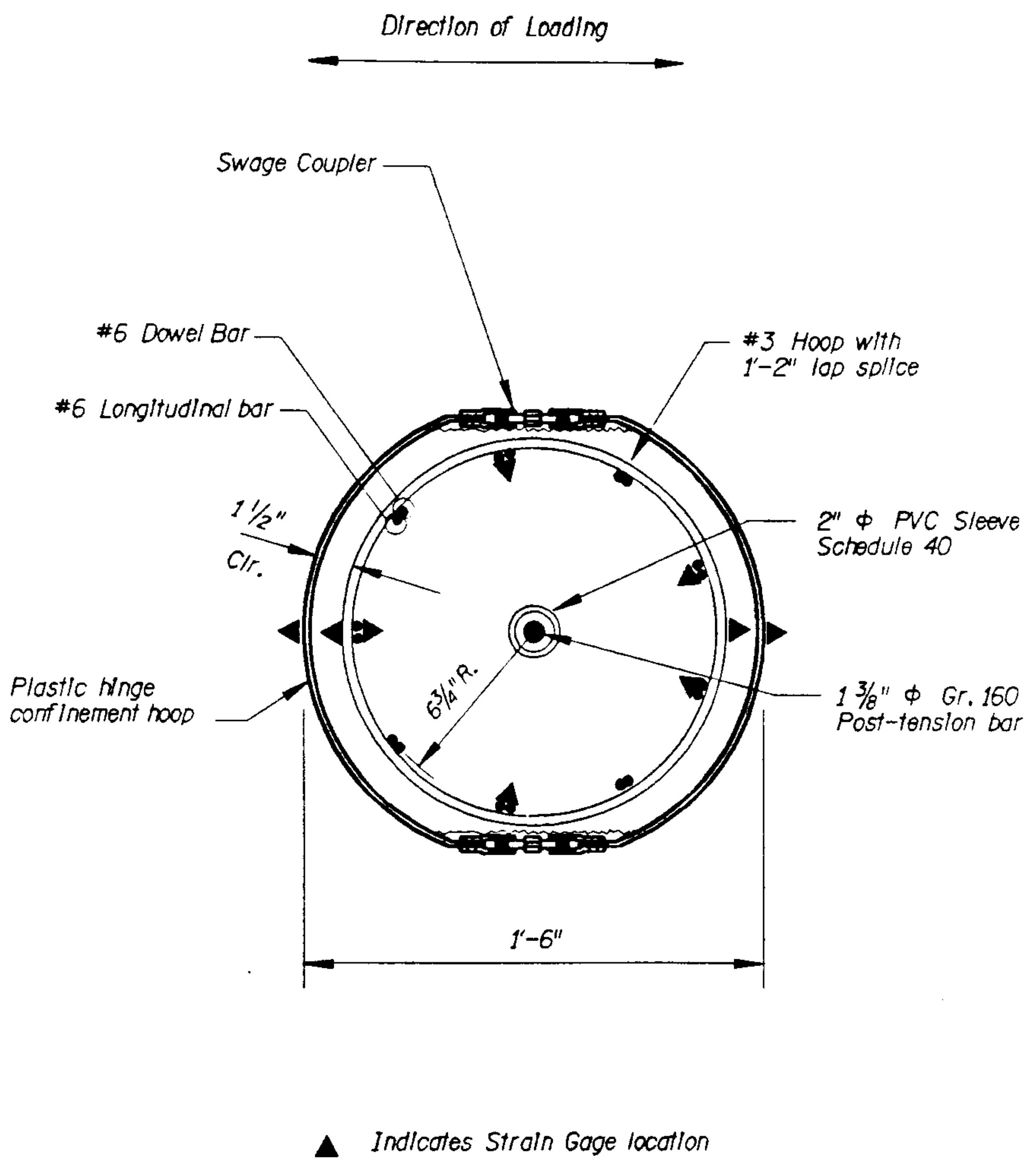

Figure 6. Cross-Section of Retrofit

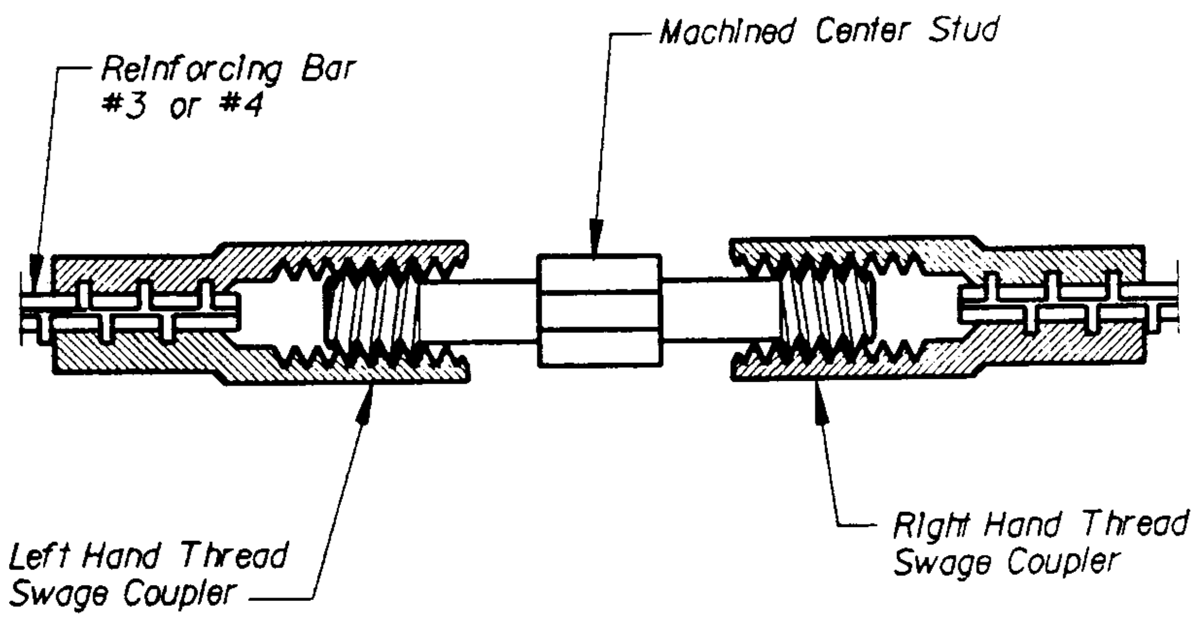

Figure 7. Swage Coupler Assembly

curing, the columns were free standing without axial load. The retrofits on columns 2, 3, and 4 were made in that state. This involved cutting diametrically opposite chord surfaces to accept the swage couplers and then placing and prestressing the hoops. The columns were then moved to the test location, and the axial load was applied just before horizontal loading The increase in hoop strain due to axial loading was measured.

The concrete mix design was based upon WSDOT standard specifications (13) for class Ax (4,000 psi) concrete. A commercial supplier produced the concrete and delivered it via transit mixer. The maximum aggregate size was 3/4 in., and the slump at delivery was 3 in. Concrete cylinders were prepared at placement, removed from forms after 10 days, and subsequently air cured with the column test specimens. The average compressive strength of 11 cylinders after 84 days was 3,200 psi, and the average modulus of elasticity was 3,176 ksi. The standard deviation of the concrete strength was 87.5 psi.

Standard deformed bars were used for the internal reinforcement (grade 40) and retrofitting hoops (grade 60). Table 2 shows the average coupon test information for yield (f_y) and ultimate (f_u) strengths.

The strength of the tested couplers exceeded 1.5 of the yield strength of #3 and #4 grade 60 retrofitting hoops.

Instrumentation and Data Acquisition

The critical measurements of the experiments were the lateral and axial loads, and the lateral tip displacement. The tip displacement was monitored by a Temposonic

Table 2. Steel Bar Strengths

Bars	Size	Grade	f_y (ksi)	f_u (ksi)
Longitudinal	#6	40	55	90
Ties	#3	40	63*	80.6
Retrofitting Hoop	#3	60	81*	99
Retrofitting Hoop	#4	60	70*	94

* Estimated values before bending

displacement transducer, which was mounted on an independent reference frame. This frame was independent of the shear panels, Universal testing machine, and the auxiliary strong floor; thus, deformations in these and in the test specimen did not affect the frame. The lateral tip load was applied by a 110-kip MTS actuator. Each end of the load train was attached to swivels, which allowed rotations about both the vertical and horizontal axes. The actuator was displacement controlled by computer. The stroke length allowed displacements up to ± 5 in. An MTS load cell placed integrally with the load train measured the force. Vertical loads were controlled manually by an electric hydraulic pump to maintain a force of 158 kip throughout the tests.

Additional to the above, strain gages were placed as in Figure 5. These consisted of gages on the longitudinal reinforcing bars and dowels at 3, 12.75, and 20.25 in. above the base plate, on the lower three internal ties, and in a like manner on the retrofitting hoops.

Rotations of the 5-in. base plate about an axis normal to the lateral load vector were also checked by vertical displacements of the top of the plate relative to the auxiliary strong floor.

All data were recorded by an HP 3497 Data Acquisition/Control unit, which interfaced an HP 9216 series 200 computer on which data reduction occurred. The data were stored on 3 1/2-in. HP 9121 disk drives. Voltage readings were normalized and zeroed relative to those measured at the start of testing. Compilations of data were completed under static conditions at the completion of each incremental lateral displacement.

Loading

First the column was fixed to the auxiliary strong floor, then, in the case of columns 2, 3, and 4, the retrofit hoops were installed, and then an axial load of 158 kips and lateral cycling were applied until defined failure. The control column required a tip lateral displacement of 1.07 in. to cause first yield of the longitudinal reinforcement.

Subsequent displacements of all specimens were defined as a multiple of this value; namely, the ductility ratio, μ. The loading sequence for all columns was one cycle at $\mu = 0.75$, two cycles at $\mu = 1$, then two cycles at $\mu = 2$, and finally continual cycling at $\mu = 4$ until failure. These arrangements are shown in Figure 8.

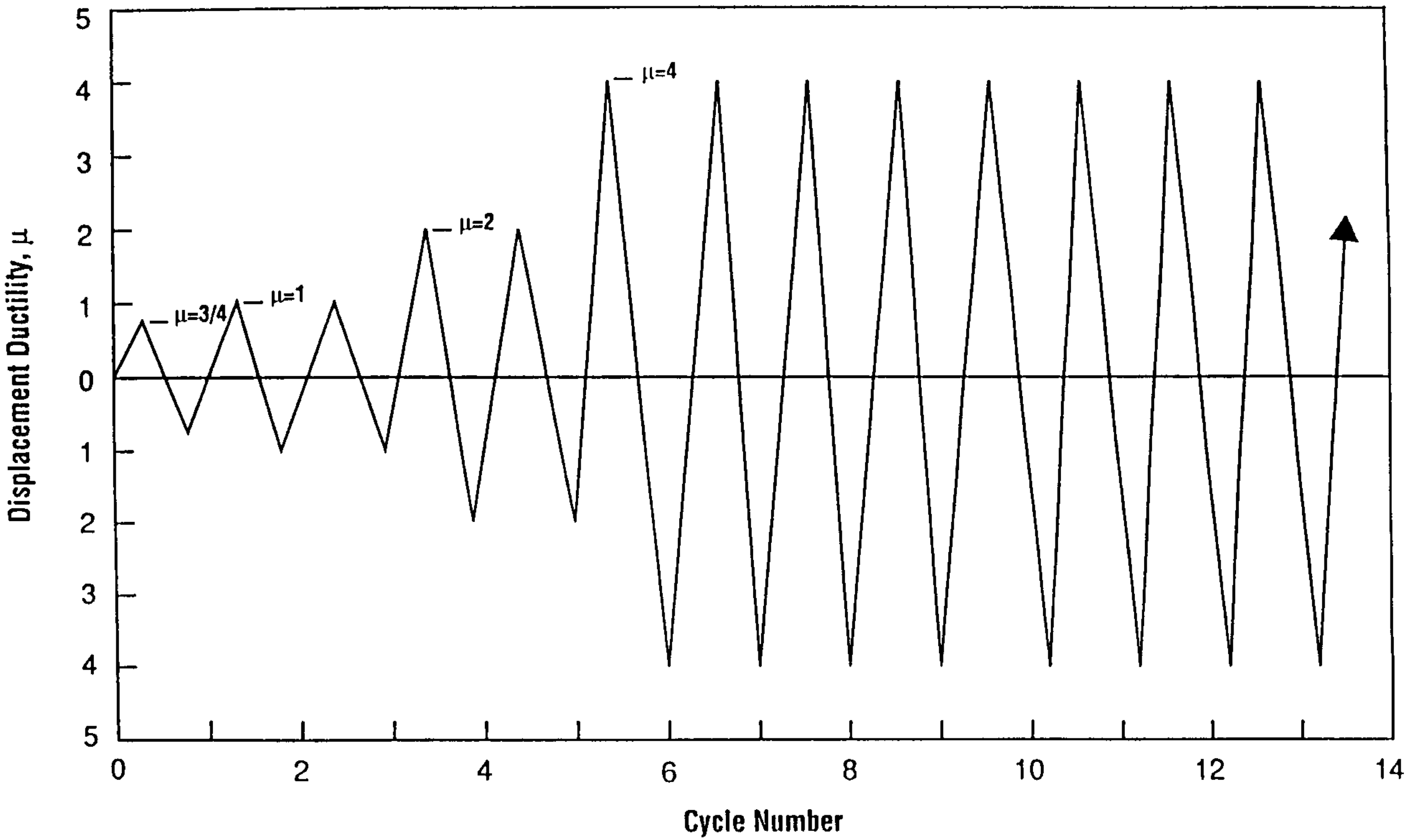

Figure 8. Horizontal Loading Sequence

FINDINGS: COLUMN BEHAVIOR

This chapter describes how the tip force-tip deflection measurements provided dissipation, stiffness, and structural integrity characteristics. Additionally, the mode of failure provided insight into the behavior.

CONTROL COLUMN (1)

Figure 9 shows the plot of the lateral tip load versus lateral tip deflection in terms of the ductility ratio, μ. Figure 10 shows the column at the completion of testing. The cover concrete spalled over the entire lap splice length. External distress became evident at 2 μ, with the enlargement of a flexural crack at the top of the bar splice and with concrete crushing near the base. The flexure crack increased in size and coalesced with other cracks to form one large diagonal crack as the displacement approached 4 μ for the first time. At this time, about 1/8 in. of crack offset was apparent at the top of the splice at the extreme tension face. The offset was the translation of the face at one side of the crack relative to the other. Also at this time, the cover concrete on the splice began to spall and increased slippage in the splice occurred. Figure 9 shows that the force needed to hold the displacement decreased. The return stroke of the first 4 μ cycle completed the hinging at the base. Subsequent cycling required smaller forces to attain the 4 μ state. In the final cycle the axial load was removed. At the completion of testing, the concrete core remained intact and was still capable of carrying the axial load. Figure 11 indicates the strains in the bottom tie of the column as the test proceeded.

The dowel strain gages 3 in. above the base indicated that tensile yielding had occurred. Only one of these gages was lost after it indicated a tensile strain in excess of 1 percent, or roughly 5 times the yield strain. The next highest maximum tensile strain was 0.18 percent. Strains were also recorded for both the dowels and the longitudinal bar gages at 12 3/4 in. above the base near the middle of the splice. The gages on the two dowels and the two longitudinal bars with the smallest effective depths for positive

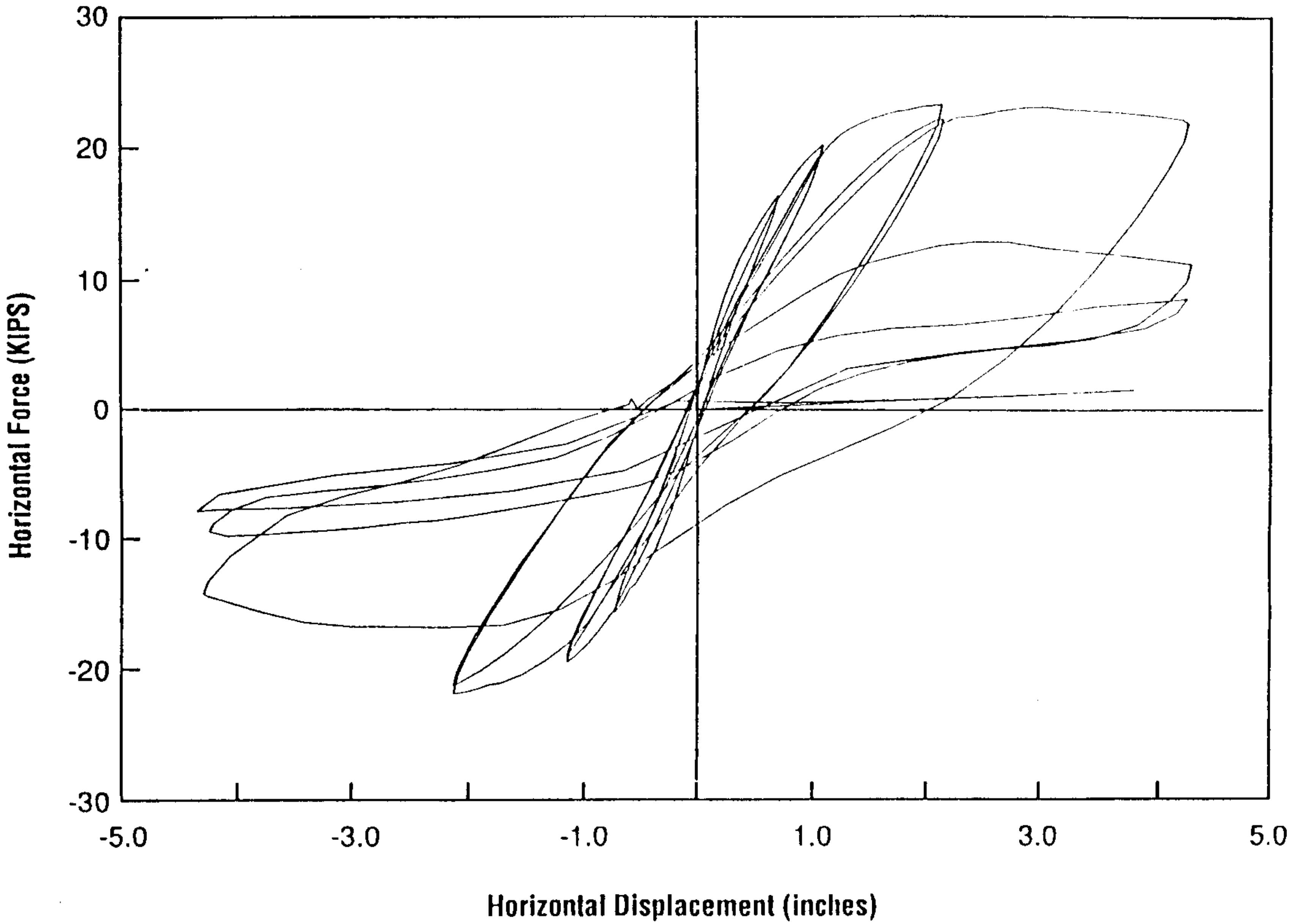

Figure 9. Column 1 — Plot of Tip Horizontal Force versus Tip Horizontal Displacement

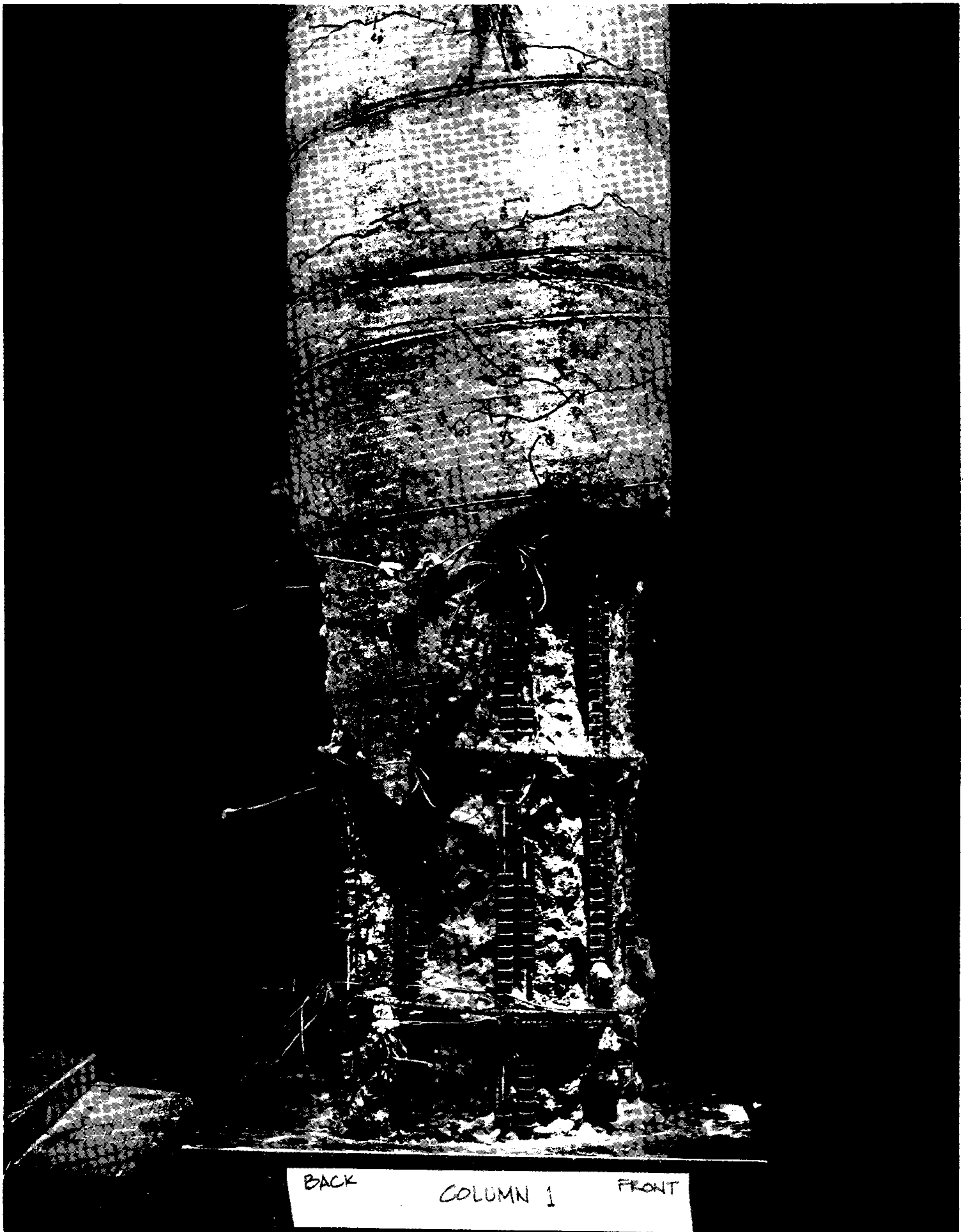

Figure 10. View of Column 1 After Testing

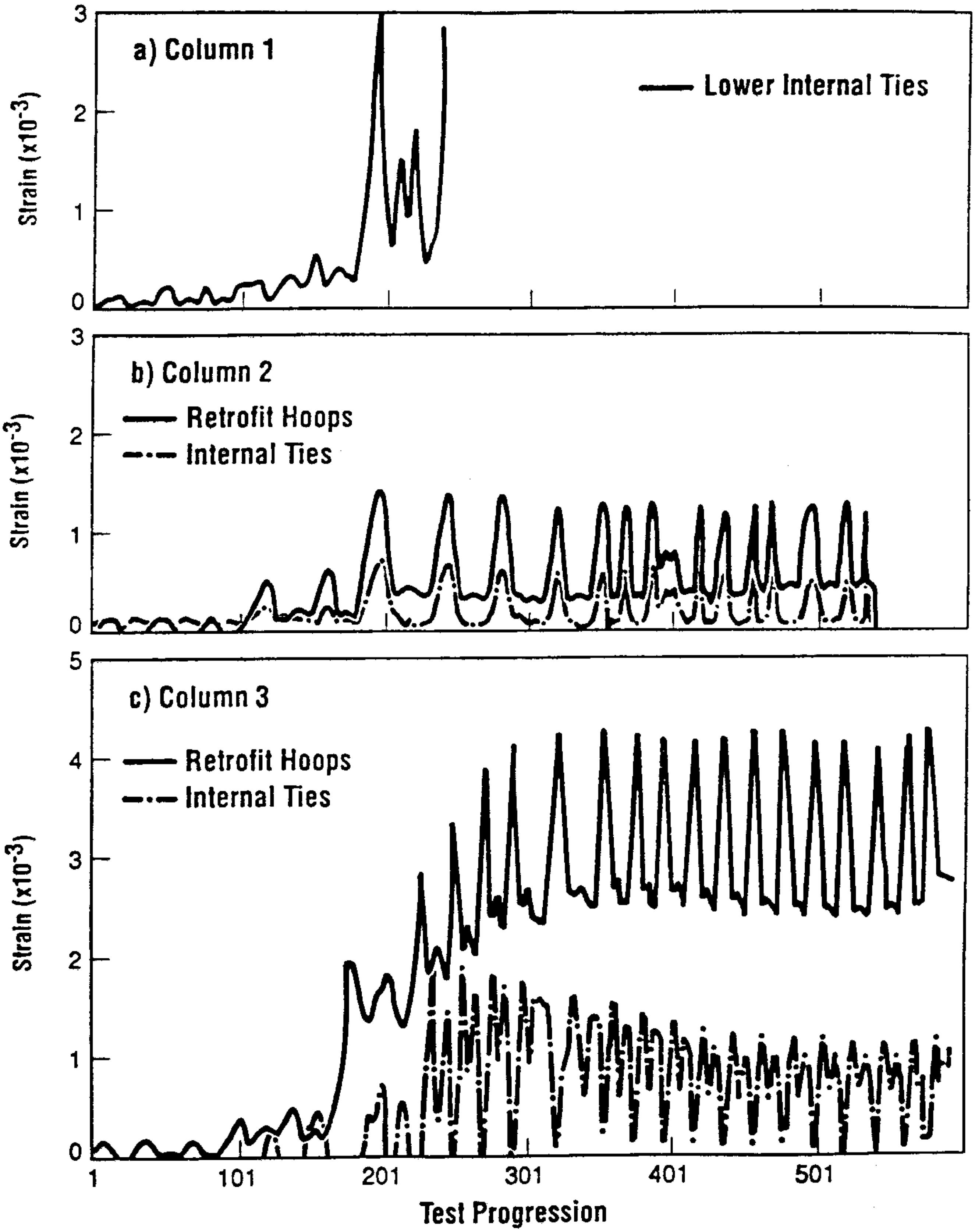

Figure 11. Strains in Retrofit Hoops and Internal Ties

displacement indicated that compression yielding occurred during loading to $\mu = +4$. These compressive strains were partially recovered in tension on the corresponding cycle to $\mu = -4$.

RETROFITTED COLUMNS (2, 3, 4)

Figures 12, 13 and 14 again show the tip load versus tip deflection plots for retrofitted columns 2, 3, and 4, respectively. The number of 4 μ cycles completed were 12, 14, and 16 for columns 2, 3, and 4, respectively. Figures 15 and 16 show photographs of columns 2 and 4 at the completion of testing. Column 3 behaved in much the same manner as column 2. Dowel reinforcing failed at the end of the cycling in the weld to the base plate. A dowel fractured at the top of the weld in column 2, and the concrete crushing extended into the core. Concrete crushing near the base occurred in columns 2 and 3 before cycling to 4 μ began, and the damage increased as the full displacements were approached. These columns also had several cracks that opened at the top of the lap splices. However, in contrast to column 1, no large shear displacements were evident. Column 4 developed a flexural crack in early cycling at the same location, and during later testing, this crack widened and exhibited a small, but noticeable, shear displacement.

All the dowel strain gages at 3 in. above the base, except for two on column 4, were rendered inoperable after indicating tensile strains in excess of 1 to 3 percent. These large strains always occurred on the cycles of displacement to $\mu = \pm 4$. The two strain gages on dowels at mid-depth of column 4 that were not lost indicated maximum strains of 0.6 percent and 0.8 percent, or roughly 3 to 4 times the yield strain. Of the five dowel gages and five longitudinal bar gages on the three retrofitted columns at the 12 3/4-in.height, seven indicated yielding. Five of the seven indicated that compressive yielding occurred first, with the maximum compressive strains at 0.4 percent, or twice the yield strain. Three of these seven gages indicated yielding only on one or two cycles. The

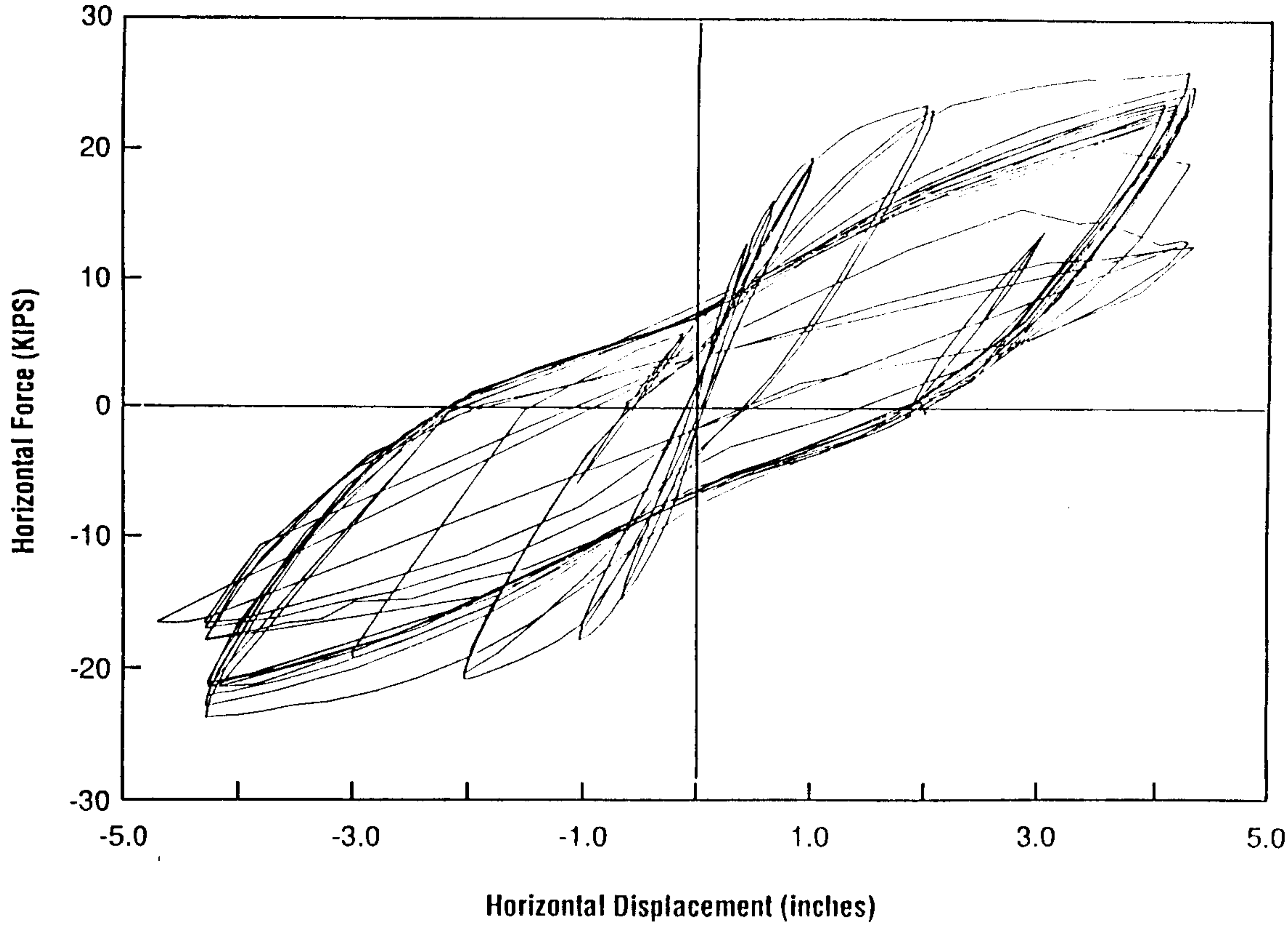

Figure 12. Column 2 — Plot of Tip Horizontal Force versus Tip Horizontal Displacement

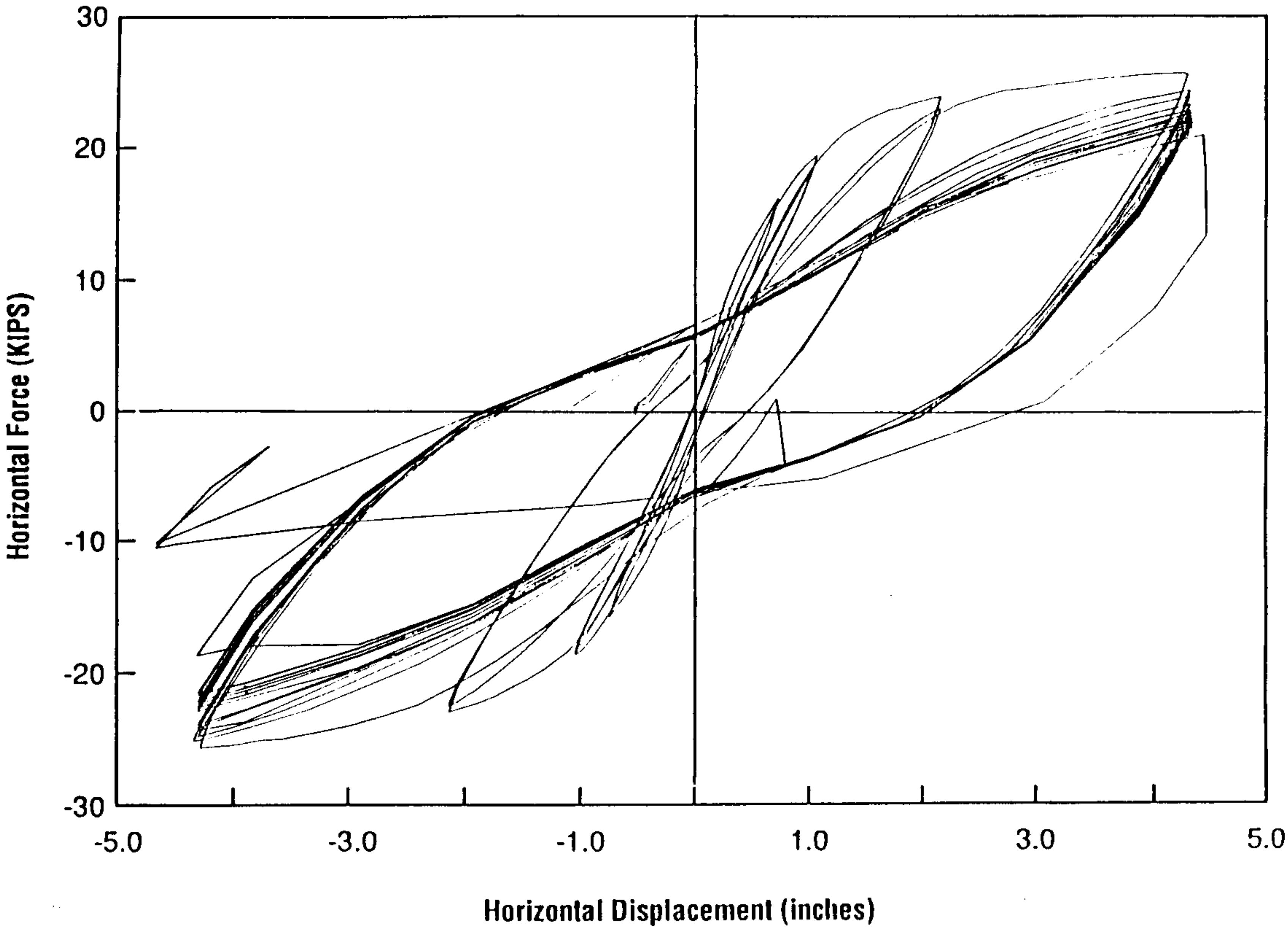

Figure 13. Column 3 — Plot of Tip Horizontal Force versus Tip Horizontal Displacement

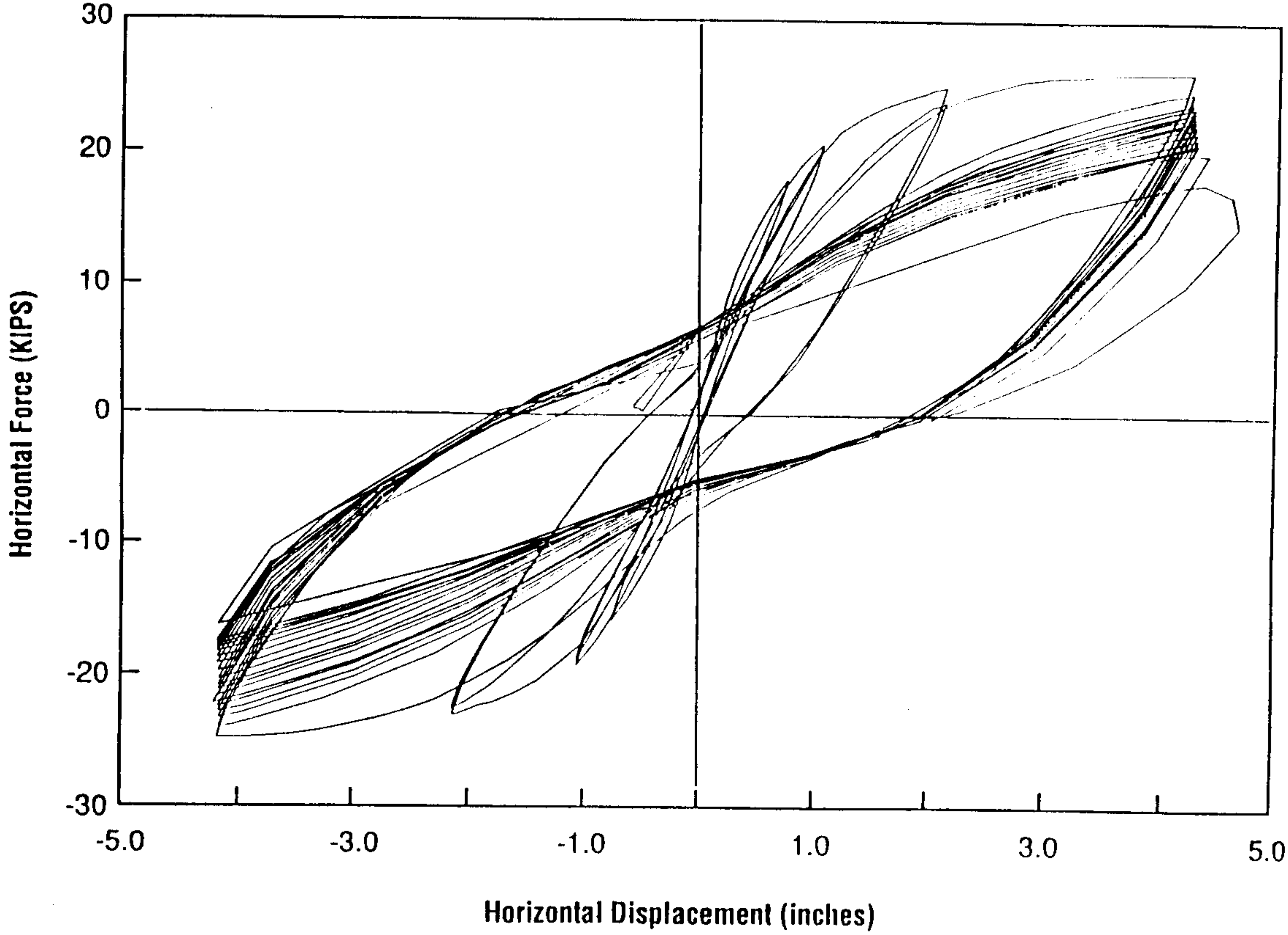

Figure 14. Column 4 — Plot of Tip Horizontal Force versus Tip Horizontal Displacement

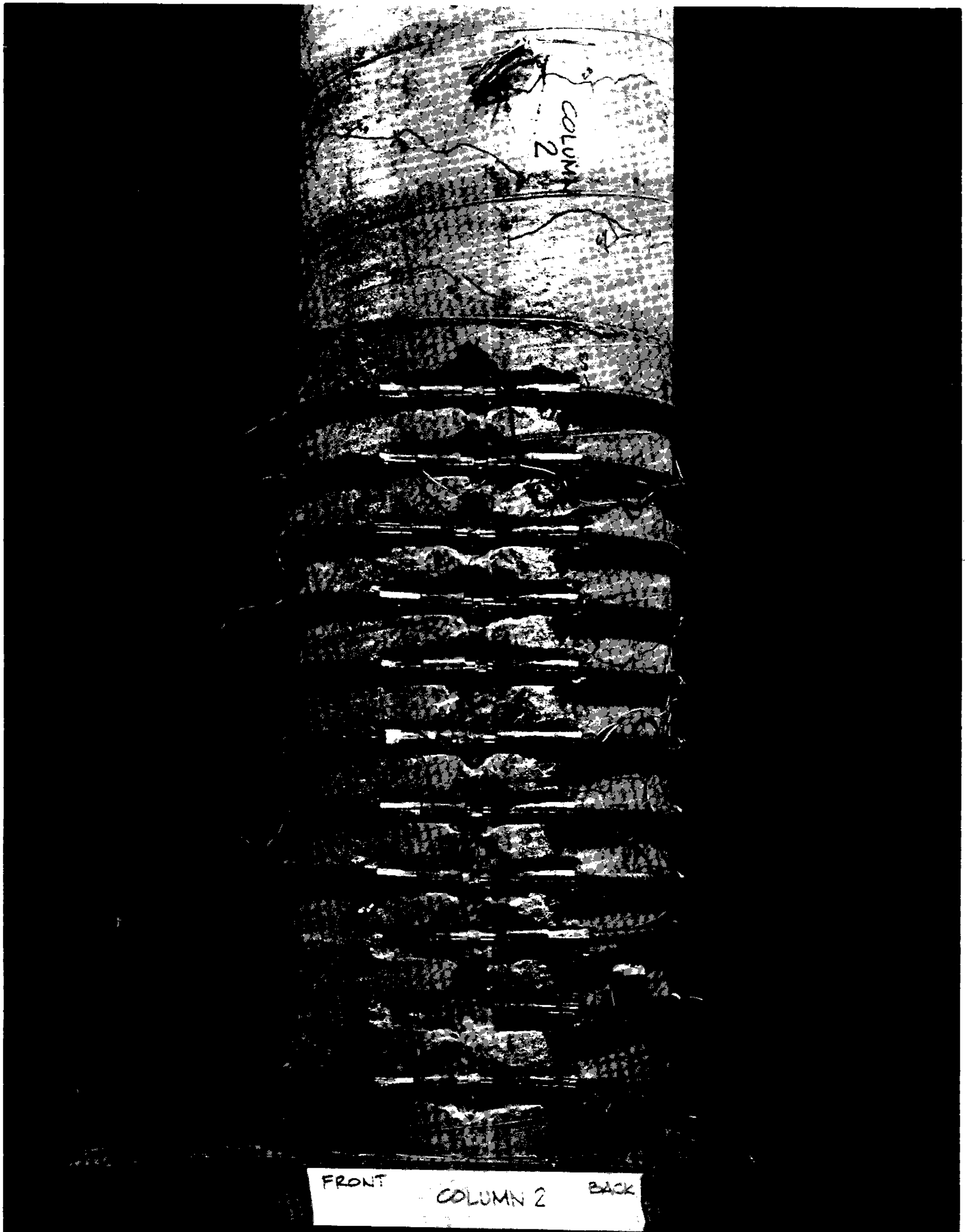

Figure 15. View of Column 2 After Testing

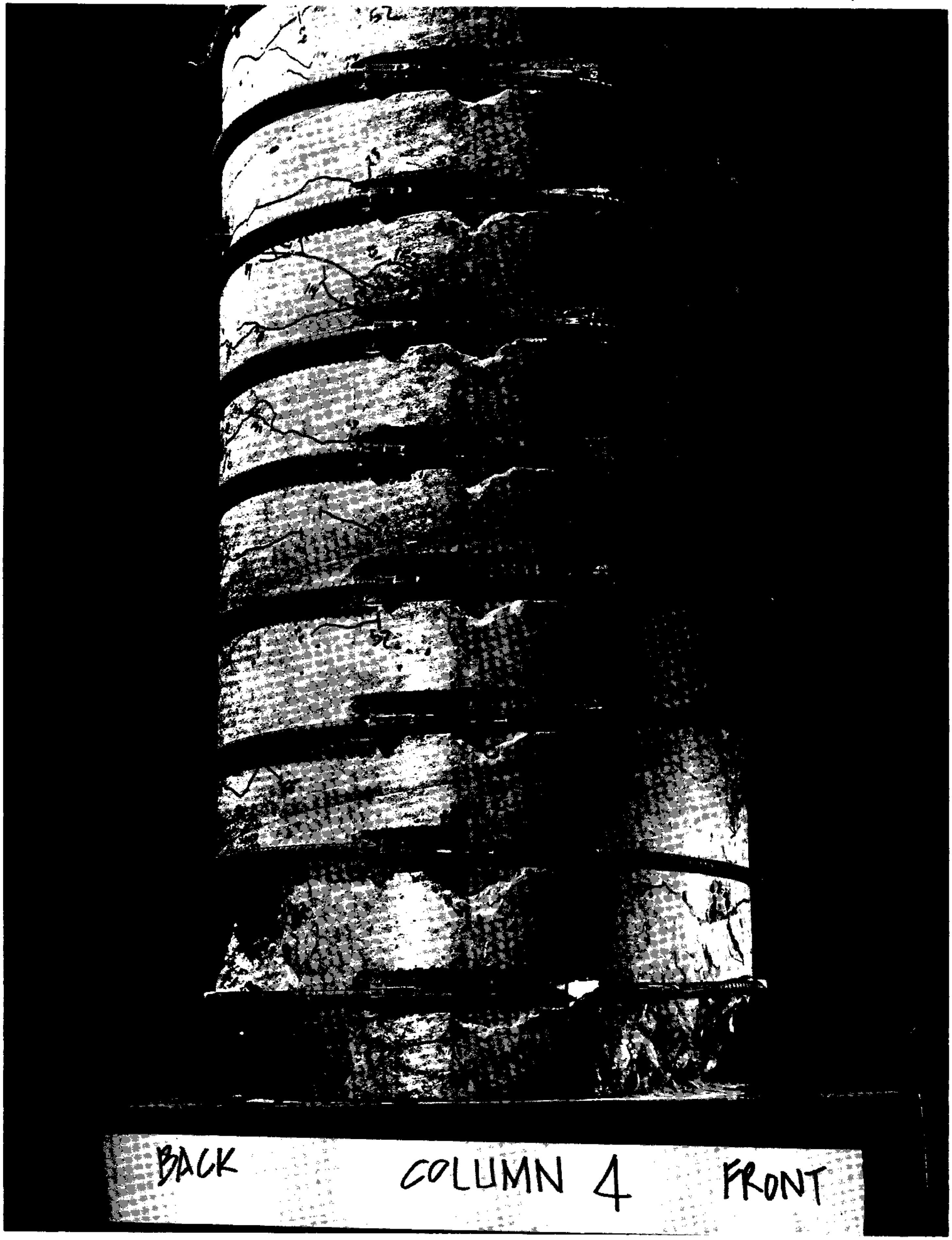

Figure 16. View of Column 4 After Testing

other four gages indicated that yielding occurred on all $\mu = 4$ cycles. However, none of the gages at mid-height of the splice were lost, and none of them indicated large (> 1 to 2 percent) strains.

Column 4 also experienced concrete crushing at the base, which propagated above the bottom retrofit hoop during the initial cycles at 4 μ. Figure 16 shows the damage to the cover concrete above this hoop at the completion of testing. In the retrofitted columns, flexural cracks formed underneath or immediately adjacent to the external hoops.

Figure 11 shows the strains in the bottom ties and retrofitting hoops for columns 2 and 4 as the tests proceeded. The ties and retrofitting hoops were more strained at locations near the tops and bottoms of the longitudinal splices. Retrofit hoops above the splices were little affected by the cycling.

INTERPRETATION AND DISCUSSION

The results are now considered in light of the objectives of the study; namely, to ascertain the effects of retrofitting on the seismic performance of bridge columns. To behave successfully under seismic cycling, the three criteria of stability of energy dissipation, preservation of stiffness, and preservation of strength have to be satisfied. In this situation, where the cycling maintained the displacements, these criteria were satisfied if the load-displacement loops of Figures 9, 12, 13, and 14 continued to overlay one another with each successive cycle. The number of cycles for which the overlay was maintained was a simple measure of satisfaction with respect to these criteria.

ENERGY DISSIPATION

Figures 9, 12, 13, and 14 show the tip lateral force-tip lateral displacement plots of the four columns tested. The test program called for a $\mu = 0.75$ cycle and then two cycles at $\mu = 1$ and at $\mu = 2$ before the continuing cycling at $\mu = 4$. The regime before $\mu = 4$ modeled some low-level lateral loading, while the $\mu = 4$ cycling replicated more intense seismic shaking. The energy dissipated in each cycle is the integration of the area contained in the respective closed loops in these plots. Figure 17 shows the value of this integration in each cycle and provides the measured energy dissipation for each cycle from the beginning of the experiment on each column. In the cycles at $\mu = 0.75$, $\mu = 1$, and $\mu = 2$, all of the columns performed in much the same manner. This suggests that the columns were indeed originally alike and that the effect of retrofitting did not alter the behavior over this range. For cycling at $\mu = 4$, the control column (1) sustained only one cycle, whereas the retrofitted columns (2, 3, 4) had loops that adequately overlaid one another for at least 12 cycles. Quantitatively, the retrofitted columns dissipated 20 percent more energy in the first $\mu = 4$ cycles than the control column; subsequent cycling in retrofitted columns reduced these initial values to approximately that of the control column. The conclusion is that the retrofitting used ensured an increase in seismic durability, as measured by the

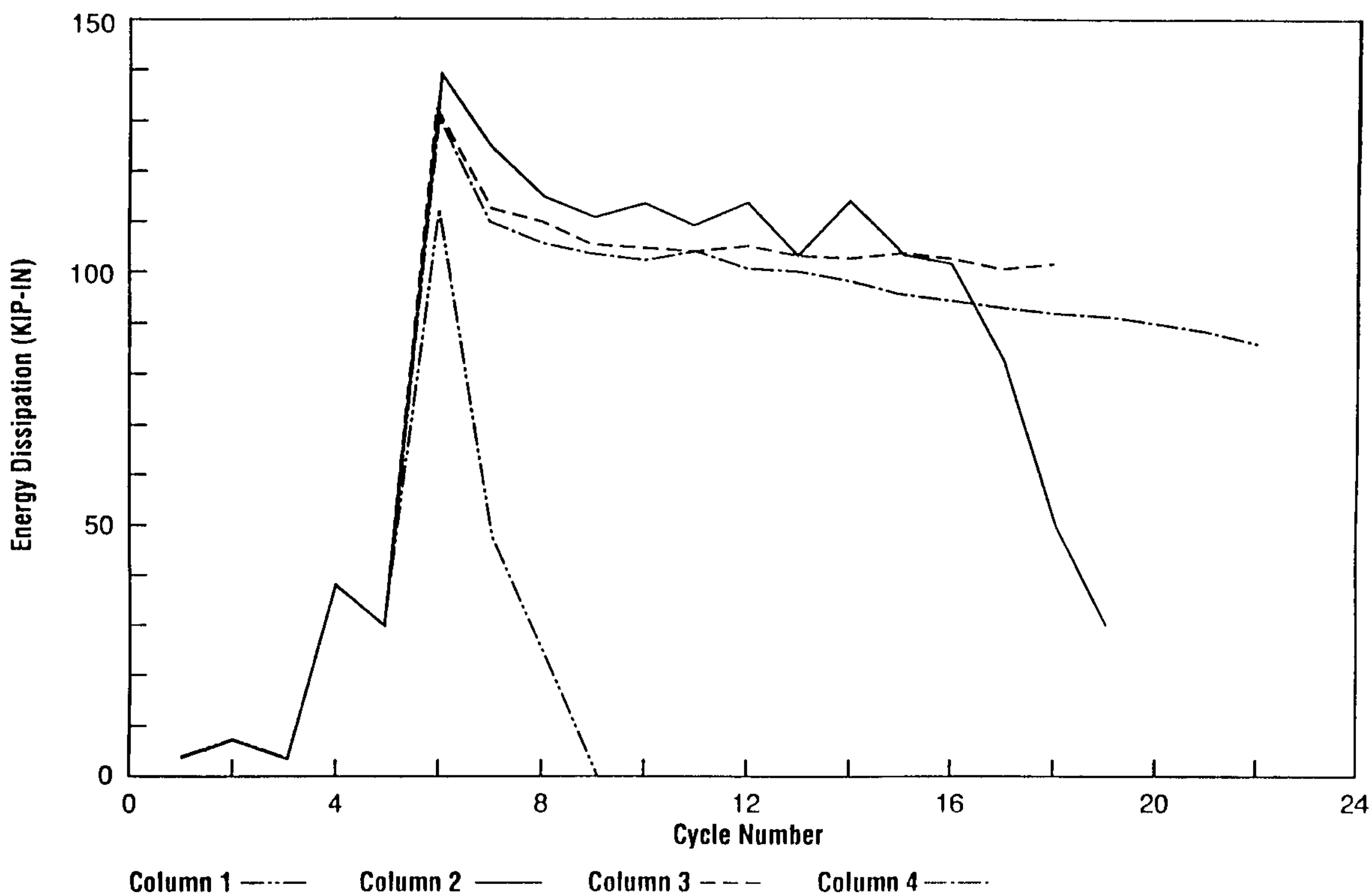

Figure 17. Energy Dissipated During Each Cycle

number of sustained cycles at $\mu = 4$ before a loss of structural integrity in the columns, by an order of magnitude over the control column performance.

The dramatic effect of retrofitting was also reflected by the total energy that each column dissipated. These totals are given in Table 3, along with the ratio of energy dissipated to that of the unretrofitted column. The total energy dissipated varied according to the hoop sizes and spacing of the retrofitting.

Evidently, the various retrofits affected this quantity in such a way that the smallest hoop size, combined with the greatest hoop spacing, produced the highest energy dissipation. Thus, at zero retrofit (column 1), the total energy dissipated had the smallest value, and at the minimum retrofit (column 4), the total energy dissipated had the greatest value. An increase in the retrofit material (columns 2 and 3) provided total energy dissipated values between these extremes. At some retrofit state defined as the hoop material per unit column length, less than or equal to that of column 4, a maximum on the total energy dissipated may exist.

STIFFNESS AND STRENGTH

The plots of Figures 9, 12, 13, and 14 indicate that at $\mu = 0.75$, $\mu = 1$, and $\mu = 2$, the stiffnesses were the same in all columns, but the cycle at $\mu = 2$ produced damage that softened the subsequent responses of the columns. Thereafter, at $\mu = 4$ cycling of the columns, the stiffness was maintained until the structural strength was lost. This occurred in the first such cycle in the control column (1) and in cycle 12 in retrofitted column 2, cycle 14 in column 3, and cycle 17 in column 4, or as follows:

$$1 : 12 : 14 : 17 \qquad \text{Equation (1)}$$

Table 3. Column Energy Dissipation

Column	Total Energy Dissipated (in-kips)	Normalized Energy Dissipation
1	269	1
2	1,459	5.41
3	1,565	5.81
4	1,842	6.84

Again, this is evidence of an optimum amount of retrofit material required with respect to the maintenance of structural stiffness and strength in this quasi-static testing regime.

The force-displacement plots show that the strength of the columns deteriorated gradually with each successive cycle to $\mu = 4$. The rate at which this degradation occurred is evident in Figure 18, which is a plot of the maximum tip force attained at each extreme positive and negative displacement. All three of the retrofitted specimens lost strength at roughly the same rate, and as the dowels began to fracture, the strength loss was dramatic and rapid.

DISCUSSION

The results of the experiments leave little doubt about the useful effect of column retrofitting on the energy dissipation and prolongation of structural life under seismic shaking. The experiments used an imposed cyclic ductility ratio of $\mu = 4$, which was intense enough to include the effects of earthquakes anticipated in the Puget Sound region. The improvement in performance under this regime suggests that the retrofit will provide a generic improvement in seismic safety. The various experiments indicate that the amount of material can be optimized; however, such a refinement is not justified in the light of the uncertainty in the seismic input.

Paulay et al. (14) observed that the yielding of reinforcement in columns subjected to loadings similar to those of the experiments reported here was restricted to a small length adjacent to the footing enlargement. Figures 12 and 13 show a limited spalling, which increased in column 4 (Figure 13) beyond that observed in columns 2 and 3. The strain gage readings on the dowel reinforcement showed that large strains apparently occurred in the spalled region. This suggests that the retrofitting produced a different spalling mechanism from that in the control column. In the control column, a conventional view attributes spalling to the bursting forces developed along the splice; with retrofit, these

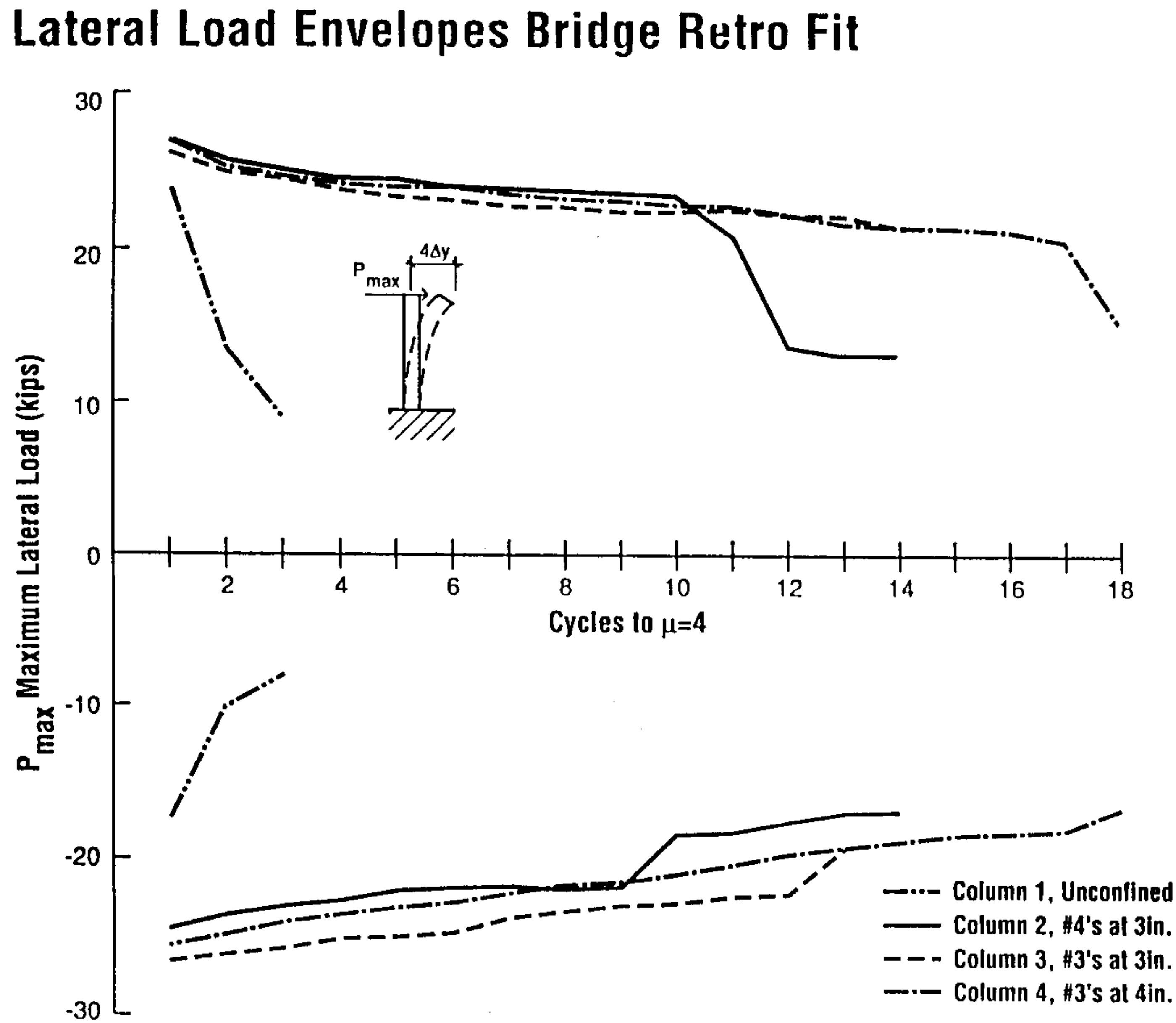

Figure 18. Applied Lateral Load Envelopes

bursting forces were resisted by the hoops. Subsequently, spalling was due to the exhaustion of concrete compression capacity.

The retrofitting appeared to enhance the force transfer between the dowels and longitudinal bars in the splice area. This occurred in spite of large, repeated cycles into the inelastic range. Such a retrofit process preserved the moment capacity in this region of the column. Additionally, the total energy dissipation was dramatically increased in the retrofitted columns above that of the control column.

The behavior of the plastic hinge in the retrofitted columns may have been different from that typically expected from conventional plastic hinge regions. Two factors contributed to this: the presence of the splice, and the location of the hoops relative to the longitudinal steel.

The proximity of the splice to the foundation means that a plastic hinge that forms at the base of the column must interact with the force transfer process of the splice. In the presence of active confinement from the retrofit, the force transfer mechanism between bars appears to be preserved. Thus, most yielding is restricted to the region near the end of the splice adjacent to the foundation. This hypothesis was supported by the damage pattern recorded in this experiment and the behavior reported by Paulay. (14)

The retrofit process used in this experiment placed initially stressed, and hence active, confinement at the outer circumference of the column. This arrangement provided confinement for both the core and the cover. Also, little or no radial dilation of the concrete was required to activate the confining elements. Thus, in a column with a moment gradient and a splice near the moment maximum, almost all yielding of the tensile steel will occur between the column end and some section within the splice. Furthermore, the splice provides additional compression reinforcement over at least part of the splice length. This steel will effectively reduce the compressive concrete strains over this length, and it will reduce the curvature produced by a given moment. The well confined splice region is then more heavily reinforced than the rest of the member, but its extent only covers some fraction of the entire splice length. This leads to localized concrete damage such as spalling and high bar strains over short yield lengths, which may lead to rupture.

A refinement of the retrofit may be an extension of the yield length to include as much longitudinal steel as possible. This may be argued formally by considering a cantilever with moment-curvature law that is elastic-perfectly plastic. The relationship between the plastic hinge curvature, δ_u, and yield curvature, δ_y, is approximately

$$\beta = \frac{\phi_u}{\phi_y} = \frac{(\mu - 1)}{3 \alpha \left(1 - \frac{\alpha}{2}\right)} + 1 \qquad \text{Equation (2)}$$

where α = l_p/l,

l = the cantilever height, and

l_p = the length of the plastic hinge.

With perfect plasticity and no slippage of the bars, the l_p values are very small. However, if the bond in and around the predicted hinge deteriorate, a spread of apparent yielding occurs into adjacent cross-sections. Observed hinge lengths range between one-half and the full depth of the section. (15) Equation 2 shows that a reduction in α causes the curvature ductility ratio, β, to increase rapidly when the displacement ductility, μ, is held constant. Apparently, locating a longitudinal bar splice in a plastic hinge zone at the end of a member produces just such a reduction in α. A significant objective of the retrofit might be to lower the β value by increasing α.

The global objective is to increase the member energy dissipation capacity by the retrofit. This may be examined from Lazan's (5) work, where the total energy dissipation is

$$D = \int_V D(v)\, dv \qquad \text{Equation (3)}$$

where $D(v)$ = the energy dissipation density, and

v = a volume in the member of total volume, V.

If the energy dissipation of only the yielding steel is considered, and if plane sections and strain compatibility are maintained, it can be shown that

$$D \approx f_y \phi \sum_{i=1}^{n} \left[A_{bi}(d_i)\, L_{yi} \right] \qquad \text{Equation (4)}$$

where
f_y = the yield stress,
ϕ = average cross-sectional curvature in the plastic hinge,
A_{bi} = area at the ith bar,
d_i = depth of the ith bar from the neutral axis,
L_{yi} = the yielding length of the ith bar, and
N = total number of bars which have yielded.

Here the yielded length, L_i, appears as the important value, and the objective of the retrofit is to extend this length to increase the energy dissipation.

These arguments indicate that from various viewpoints, the success of the retrofit is associated with the ability to extend the yielded length of the longitudinal bars in the hinge region. In this way both the strength will be maintained in seismic excitation and the energy dissipation increased.

The extension of the yielded length of the longitudinal reinforcement in the hinge region has been shown to enhance both the strength and damping associated with the steel. Such yielding will not affect the stiffness of the column, and any attenuation of stiffness will be associated with the performance of the concrete. This was evident in the various hysteresis plots, where early cycling at $\mu = 2$ induced softening by reason of concrete cracking. The retrofitted columns subsequently maintained their stiffnesses and hence concrete integrity, until failure.

The indication that column 4 sustained more cycles at $\mu = 4$ than columns 2 and 3 may be connected with the performance of the external hoop retrofit. In columns 2 and 3, these hoops, as well as the internal ties, did not yield (Figure 11); however, both lower external hoops and ties did yield in column 4. This yielding of the confinement steel evidently allowed an extension of the yielded length of the longitudinal reinforcement. This extension was vicariously displayed by the increased spalling above the lowest hoop, shown in Figure 16; this spalling was absent in columns 2 and 3. The yielding of the lower external ties was an apparent advantage in the arrangement and seismic simulation described herein. However, in other retrofitted columns and seismic experience the yielding of the hoops may not be an advantage. Indeed, the quasi-static test used may have concealed the damage caused by jerkiness in the input, which could have dislodged the yielded, lowest loop and thus negated any benefit of the retrofit.

The tests did not include the true interaction between the columns and the footing. The rigid connection used, in which careful measurements showed zero base rotation, would have been softened if the footing had been included in the system.

This complete system was used by Priestley (17) in his tests on retrofitted columns; these retrofits utilized an oversized steel jacket around the column where the resulting annulus was filled with grout. In Priestley's arrangement, the retrofit may increase the column stiffness and strength; thus knowledge of the footing performance is crucial.

Inclusion of the foundation allows penetration of yielding and concrete damage into the footing. For the columns reported here, this might extend the life of the plastic hinge in terms of the number of cycles that the hinge could endure, provided another mode of failure associated with the foundation does not occur. For instance, the combination of spalling in the lower hoop interval and the deterioration of the foundation concrete just below the column-footing joint might permit the dowels to buckle in compression.

Again, it must be emphasized that the tests reported were intended to simulate the long duration, medium intensity earthquakes deemed possible in the Puget Sound area. The effectiveness of the methods may well be generic, but the evidence is only for the simplistic loading sequence used.

These experiments were deliberately concerned with columns in which flexure dominated the member response resulting from tip displacement. For columns fixed at the base and at the top cross-beam, a total height of 40 ft. was modeled in these tests. For short columns subjected to the same tip displacements, the member response will be affected by shear, and the validity of this retrofit under conditions of high shear remains undetermined.

CONCLUSIONS

Half-scale models of typical long, circular reinforced concrete bridge columns of the 1950 to 1970 era were retrofitted to enhance seismic performance and then tested to failure with the application of repeated, quasi-static lateral loads. These columns all contained a longitudinal bar splice in the region of maximum, induced moment. The retrofit consisted of circular hoops prestressed around the outside of the columns. In summary, this retrofit

- did not alter the column stiffness,
- did not significantly increase the column strength,
- increased slightly the energy dissipated in a single cycle, and
- increased by an order of magnitude the total number of cycles that could be sustained before failure.

Such conclusions provide confidence in the use of this retrofit procedure on columns of this era where shear is not a significant failure feature and where the earthquake is of medium intensity and long duration. The results suggested that the amount of material used in the retrofit affected the details but not the substance of the behavior.

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Complete test data are available in Coffman (18).

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